STUDY ON THE UNIQUE DESIGN, CONSTRUCTION AND OPERATION OF GAS PIPELINES IN THE PERMAFROST OF EAST SIBERIA

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by

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Preface

Over the past decade there has been a recognition that Asia will undergo tremendous economic growth in the early 21st century. Associated with this growth will be an enormous demand for relatively inexpensive energy sources. Much of today's energy sources in east Asia are coal (50.1%), oil (36.1%) and nuclear (6.8%), most of which are considered environmentally unfriendly. Only 5.5% of the energy in the region is derived from natural gas. Most of the gas used today is relatively expensive liquefied natural gas (LNG) from southeast Asia. There are however, vast natural gas reserves in northeast and central Asia, such as Sakhalin, Sakha Republic, Krasnoyarsk - Irkutsk Region, Turkmenistan, as well as China's coastal area and western provinces. Therefore, the concept of supplying piped natural gas (PNG) to the region has been developed in recent years.

In 1989, the National Pipeline Research Society of Japan was formed with the specific objective of investigating the feasibility of constructing a multi-national PNG pipeline network in the Asia-Pacific region, including submarine pipelines to South Korea and Japan. These studies resulted in the "Proposal on the Trans-Asian Natural Gas Pipeline Project", which has become the core element of the Asian-Pacific Energy Community. It is recognized that this PNG network is not only essential for the regions environmentally acceptable energy supply, but it is also considered that the geopolitical cooperation required for such a massive project could make a major contribution toward peace and stability in the region.

The prime reason for undertaking this study was that the Ministry of International Trade and Industry (MITI) and Japan National Oil Corporation jointly decided to make a grant of 6 billion yen to each of the Sakha Republic and the Irkutsk Oblast (Russian government) in 1995 as part of an aid grant for the investigation of oil and natural gas in East Siberia. Nippon Steel Corporation, the managing company for the natural gas section of the Russo-Japanese Economic Committee of the Federation of Economic Organizations, was requested to cooperate in the feasibility study of the construction of pipelines starting from promising gas and oil fields in these regions.

A prefeasibility study (PFS) is being made for a scheduled period of three years and is being conducted by four newly-organized task forces. They are, the Supply and Demand Study Task Force, the Infrastructure Study Task Force (both chaired by Prof. Murakami of Hokkaido University), the Tax and Law Study Task Force (chaired by Prof. Fujiwara of Keio University) and the Pipeline Construction Study Task Force (chaired by the author of this thesis, Mr. O'hashi of Nippon Steel).

As MITI and Japan National Oil Corporation hope to assume an operating life of 50 years it was necessary to conduct a detailed research and analysis of problems with pipelines in permafrost terrain. The concern was primarily related to frequent reports of damage and trouble with pipelines in West Siberia.

In Japan, the author has obtained the advice of Prof. Fukuda of Hokkaido University, who is called the first man in Japan in the study of the behavior of permafrost layers, and the general guidance of Prof. Ono at the Engineering Research Course of the Postgraduate School of Kyoto University who is working for the National Pipeline Research Society of Japan as vice-chairman.

The author has been involved in consultations with other experts and obtained the cooperation of Dr. John Heap, professor at Scott Polar Research Institute, affiliated to Cambridge University in the United Kingdom, who has a detailed knowledge about the northernmost regions; Dr. P. J. Williams, professor at Carleton University in Ottawa, Canada, who is well known for his study of frost heaving; AGRA Engineering, Construction & Technology, a Canadian engineering company with considerable experience in the permafrost engineering design and construction for arctic pipelines in North America and Russia, and Cincinnati University, who are well-informed on American major pipelines.

There is of course limited experience with pipelines in permafrost in Japan. Many Japanese researchers have studied the freezing and thawing of soils, but not so much in the context of pipelines in permafrost. The author, in his capacity as chairman of the Pipeline Construction Study Task Force, quickly recognized that the design and construction of pipelines in permafrost would be the major challenge to the feasibility of any gas pipeline from East Siberia to the Far East. In addition to reviewing the problems with the Russian pipelines, it was necessary to determine the present international capability for the preparation of reliable designs, to carry out the construction in an environmentally acceptable and economic manner, and to safely operate large diameter gas pipelines in permafrost.

Based on numerous consultations, site visits and an extensive review of available books and published papers, the author has been able to summarize the available international expertise and apply such expertise to assess the feasibility of constructing a large diameter gas pipeline from Ust'-Kut in the Irkutsk region, through Mongolia to Beijing and on to Tianjin on the Yellow Sea coast.

This study has found that the permafrost regions are generally divided into two main sub-regions - the colder "continuous" permafrost in the higher arctic and the "discontinuous" permafrost in the mid-arctic and sub-arctic. In eastern Asia, the "continuous" permafrost is found to the north of Yakutsk. The "discontinuous" permafrost extends southward from there for as much as 2000 km, into the Mongolian mountains. The area of interest for the Ust'-Kut to Tianjin pipeline lies entirely within the warmer, southern portion of the discontinuous permafrost. Approximately half of the pipeline route is underlain by intermittent permafrost.

This thesis presents the considerable background of information available on existing pipelines in permafrost and numerous design studies and test facilities - in Russia and North America. The available Russian information is quite limited compared to the extensive system of pipelines in permafrost. The North American information is more extensive as access is available to more published articles and through personal consultations with the design and construction experts involved.

Relevant details on the distribution and characteristics of permafrost are presented. The typical properties of frozen ground and its behaviour on thawing are discussed. This sets the technical background for the subsequent presentation on the design approaches that have been developed over the years to mitigate the negative impacts of pipeline construction and operation on the permafrost. It is recommended that these approaches be applied to the proposed pipeline from East Siberia.

This study has found that the greatest design issues for pipelines in the discontinuous permafrost region are, thaw settlement of the pipe, the stability of thawing permafrost slopes, and permafrost foundations. There are also construction and operations constraints related to the sensitivity of the permafrost terrain to disturbance. The author has specifically reviewed the expertise gained through major research and design studies conducted in the 1970's and 1980's, for large diameter arctic pipelines in North America. In addition the author has visited, and closely studied the design and construction approach used for the Norman Wells oil pipeline, the first fully buried pipeline in North American permafrost. Several consultations were held with the owners and operators of this pipeline, and with the engineers responsible for the design, construction and monitoring of the pipeline.

The main contribution of this thesis is the application of the available expertise that the author has learned, to the specific feasibility study for the Ust'-Kut to Tianjin gas pipeline. The basis for the selection of the route is presented. The specific permafrost conditions along the northern portion of the route are summarized. Borehole data is presented for the most northerly permafrost region.

Design details are provided for the hydraulics, the thaw settlement and associated pipe structural design. Based on the available data, this study has shown that the pipeline can be designed for up to 1.1 m of differential settlement, based on API 5L X-70 pipe and 22.2 mm wall thickness. This allowable thaw settlement coincides with the predicted thaw settlement for a cooled pipe temperature of 10°C. This cooling is necessary, as the more normal operating temperature of about 40°C would cause too much thaw settlement. This pipe structural design is based on established procedures permitted in the American ASME B31.8 design code, which is widely accepted for international pipelines. This code allows for plastic strain criteria for secondary loading on the pipe such as thermal expansion, and thaw settlement and frost heave displacements. Design engineers have applied this approach to the design of several pipeline projects in permafrost, and in particular to the Norman Wells pipeline.

According to this study it has been established that the pipeline can be buried in the permafrost region. This design mode is considerably cheaper than an above-grade alternative. As a result of this study the author has concluded that, enabling the buried mode, will be the single most important factor in realizing an economically feasible pipeline project.

The other major design concern relates to the stability of thawing ice-rich slopes. The analyses presented herein demonstrate that ice rich slopes in the most northerly portion of the route can be designed to remain stable if adequate insulation is applied to the slopes. Based on the successful experience in North America, the use of natural wood chip insulation is recommended for the proposed pipeline.

According to this study the cooling of the gas to reduce the thaw settlement in the permafrost region, will require the installation of refrigeration or coolers at most of the northerly compressor stations. Nineteen compressor stations in all are recommended, with 15 requiring some degree of cooling. Foundation design for the northern facilities must be based on frozen ground engineering approaches. Since the permafrost is warmer than -1°C, the study recommends the installation of ground-cooling thermosyphons, such as were used on the Alyeska pipeline supports, to ensure critical pile foundations remain sufficiently frozen to maintain integrity.

The capital and operating costs for the proposed pipeline have been estimated and the costs are compared with North American statistical information. Financial analyses have been performed to illustrate the cost of transportation for the natural gas through the pipeline system. Several scenarios are considered.

The environmental constraints unique to the pipeline route are identified. The Lake Baikal region is a declared environmental protection zone. All permafrost terrain is sensitive, especially where ice rich conditions exist. According to this study, thawing beneath the construction right of way will be initiated simply by clearing the trees in advance of construction. Winter construction is proposed as the main approach to minimizing permafrost terrain disturbance.

The basic construction plan and schedule for this Russian pipeline are presented. As a result of this review, it is demonstrated that the pipeline can be constructed in three years of pipe laying. The procedures that have been developed through many years of North American arctic construction are summarized. Some of the more unique challenges are discussed and the appropriate procedures are proposed.

Recommendations are made for the operations and maintenance program that will be required to maintain a safe operation. There will be the potential for drainage and erosion problems in the early years following construction. Also, there will be a need for a reliable plan for monitoring pipe settlement and slopes performance. The study has established that the development of differential pipe settlement can be detected in advance by the latest "intelligent" pigging devices. The slopes performance will require regular monitoring by temperature and porewater pressure sensing instrumentation.

In conclusion, the author has determined that the required expertise and experience from North America for the design, construction and operation of a gas pipeline in such permafrost conditions can be applied to the proposed pipeline from East Siberia. The author's study has established that a natural gas pipeline can be constructed from Ust'-Kut to Tianjin in a technically, environmentally and economically feasible manner.

Acknowledgments

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF THE STUDY

The demand for piped natural gas (PNG) as an energy source in the Far East, as opposed to the environmentally unacceptable coal, oil or nuclear based energy or the more expensive LNG, is creating considerable interest in the gas reserves in Eastern Siberia and other central Asian countries. A network of natural gas transmission lines has been considered for several years. Various feasibility studies have been conducted. For example, the Japan National Oil Corporation has recently commissioned a pre-feasibility study for two potential gas pipelines from the Irkutsk region in East Siberia to the Far East (ESFE). These options included a pipeline to Tianjin, China, via Mongolia; and a pipeline to Vladivostok, following the Trans-Siberian railway.

Any one of these large diameter pipeline projects would be a major undertaking and will involve several countries as producers and consumers, as well as those countries through which the pipeline would pass. In addition to the multi-national geopolitical and economic challenges of such a project, there is also a significant technical, engineering challenge. The terrain conditions along the northern parts of these East Siberian routes includes warm, discontinuous permafrost. There is generally less than 50%, intermittent permafrost occurrences over most of these northern sections. The main issues for pipeline projects in these permafrost conditions (as opposed to conventional pipelines in southern regions) are:

- thaw settlement which can induce bending of the pipe,
- the stability of thawing slopes,
- permafrost foundations for facilities, and
- construction constraints

In the Japanese scientific and engineering communities, there is only limited experience with natural permafrost (e.g., Sone *et al*, 1988), and there are no Japanese pipelines in permafrost. There have been numerous Japanese scientists who have studied permafrost occurrences in other countries (e.g.., Fujino *et al*, 1988 and Horiguchi, 1988). The effects of frost heave and thaw settlement have been studied in the laboratory by others (e.g., Yamamoto *et al*, 1988, Akagawa *et al*, 1988 and Nakano and Takeda, 1994). However, there is essentially no Japanese experience in the practical design and construction of pipelines in permafrost.

The author has made a determined effort to learn the established, state-of-the-art approach to designing, constructing and operating major natural gas pipelines in permafrost regions. In order to accomplish this, the author has:

- visited several countries which have major natural gas pipelines,
- consulted on several occasions with North American arctic pipeline researchers and designers,
- visited, and conducted interviews with the designers, constructors and operators of a Canadian permafrost pipeline,
- reviewed the extensive relevant pipeline experience in Russia and North America,
- reviewed many books and papers relevant to permafrost pipelines,
- studied the costs of numerous major multi-national pipelines

The approaches learned from these studies are then applied to the specific assessment of the feasibility of a large diameter natural gas pipeline from Irkutsk, East Siberia to Beijing and on to Tianjin, on the Yellow Sea coast. The author will demonstrate that the design concerns and construction constraints can be mitigated and overcome, and that reliable and economic pipelines can be constructed in these permafrost conditions. It is not necessary to construct the pipeline using a very expensive above-grade mode as for the Alyeska pipeline. Furthermore, by using appropriate design expertise and construction techniques, the problems experienced with many Russian pipelines can be avoided. North American experience in particular has established that pipelines can be designed and constructed in permafrost using the buried design mode, which is much more economic than an above grade alternative. The author is able to conclude that the natural gas pipeline from East Siberia can be economic and can become a major factor in providing Japan with a competitive energy source.

It is the intention in this thesis to concentrate on those aspects of pipeline design, construction and operation that are unique to permafrost conditions. The conventional pipeline practice is only briefly reviewed where necessary, as a basis for comparison.

Sections 1 to 4 of the thesis present the economic requirement for piped natural gas to provide energy for Japan into the early 21st century, an introduction to the unique characteristics of permafrost and the approaches used in designing pipelines in permafrost. In Sections 5 to 8 the author specifically applies these approaches to study the feasibility of a gas pipeline from the Irkutsk region to Tianjin. The conclusions of the thesis are presented in Section 9, including recommendations for further research.

1.2 BACKGROUND OF PIPELINES IN PERMAFROST REGIONS

1.2.1 Russian Experience

Based on a review of a limited number of Russian papers[1] available, the following summary has been prepared. The Russian gas pipeline system consists of a total pipe length exceeding 200,000 km. The main Russian gas fields are located in the permafrost regions (West Siberia, Sakha Republic) and the total length of main gas pipelines in permafrost is almost 11,000 km. Hence, Russia has accumulated a wide experience in the design, construction and maintenance of pipelines in permafrost. Unfortunately, this experience has not been published as it was considered "secret". In recent years, the secrecy laws have been relaxed, however, Russia is now experiencing a difficult economical situation and there are no funds to compile and analyze existing experience.

[1] Throughout this thesis, many specific references are given, however, much of the information was obtained from the study of many books, reports, journals, etc., all of which are listed in the complete reference list at the end of the thesis. Much of the information was obtained through personal communications with various experts, which are also noted in the references.

1.2.1.1 <u>History of Russian Pipelines in Permafrost</u>

The first Russian oil pipeline was built in 1906 from Baku to Batumi in the Caucasus mountains and the first gas pipeline was built in 1942 - 1944 from Buguruslan to Kuybyshev in the European part of Russia. The first Russian gas pipeline in permafrost was built in 1967 from Taas-Tumus to Yakutsk (No. 1 on Figure 1.1), with a length of 500 km and pipe diameter equals 529 mm (21"). The gas pipeline Messoyakha - Norilsk (West Siberia - Taymyr peninsula - No. 2 on Figure 1.1) is built in 1969. It is the first gas pipeline constructed north of the Arctic Circle. This system, consisting of three pipes, has a length of 260 km and the pipe diameter equals 720 mm (28"). Construction of the main pipelines from West Siberia to the European part of Russia was begun in 1970. Gas is gathered from three giant gas fields: Medvezhye, Urengoyskoe, Yamburgskoe and several minor gas fields. All gas fields are connected by a system of pipelines. There are three main corridors for the main pipelines from West Siberia:

a) to the Ob crossing near Peregrebnoe (3A on Figure 1.1), with seven pipes,
b) to the Ob crossing near Octyabrskoe (3B), with 9 pipes, and
c) to the Ob crossing near Surgut (3C), with 2 pipes.

Generally, the pipe diameter is 1020 mm (40") in connector pipelines between gas production areas, and 1220 mm (48") or 1420 mm (56") for the main transmission pipelines. The construction of this system took place over approximately 20 years. The total pipe length in permafrost is approximately 10,600 km.

At present, Russia is constructing a pipeline from Zapolyarnoe gas field to Urengoyskoe gas field, with a length around 100 km and pipe diameter of 1420 mm. The Zapolyarnoe gas will raise the pressure of the gas pipeline from Urengoyskoe gas field.

In 1995, Russia has studied the feasibility for developing the Yamal gas fields. The largest gas/condensate fields are: Bovanenkovskoe (4 700 billion m^3), Kruzenshternovskoe (1 300 billion m^3) and Kharasaveyskoe (900 billion m^3). The Yamal peninsula has a unique natural environment which can be easy disturbed by a human activity. New engineering, organizational and transport design concepts have been proposed to reduce the impact of the development on the sensitive environment. First of all, the capacity of the gas process units is to be increased so that the area of expropriated lands can be reduced. The gas will be cooled to between -3 and -5°C and it is assumed that the pipeline will be buried. The majority of the facilities will be comprised of modules. There are two route options for the most northerly section of the pipeline system (to Compressor Station Seida), as shown on Figure 1.2:

a) the Baydaratskaya Bay Crossing option of 514 km length, and
b) the onshore route of 614 km around the bay.

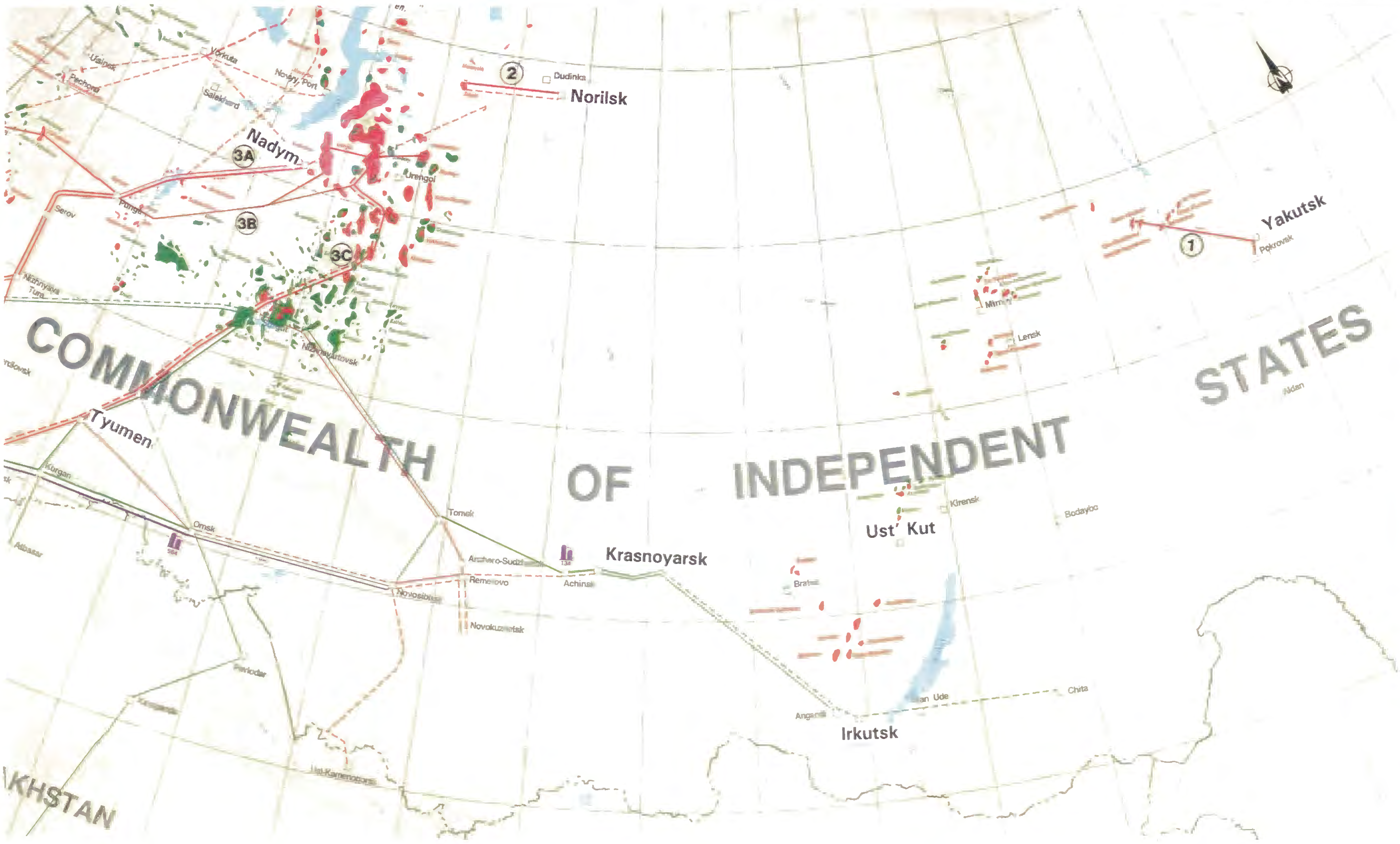


Figure 1.1: Major Russian Pipelines in Permafrost Regions

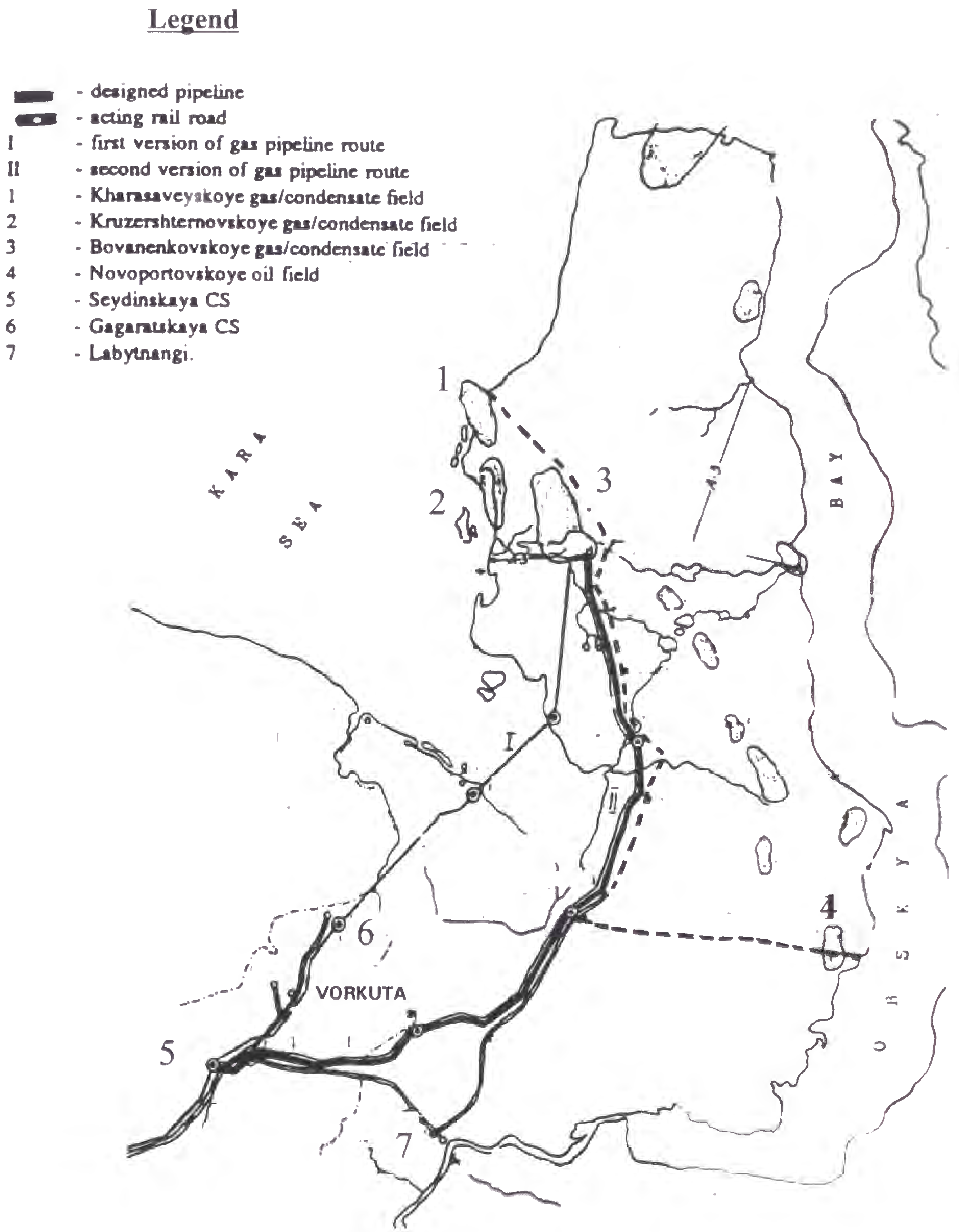


Figure 1.2: Head section of Yamal-West gas pipeline system

The length of the bay crossing option is 100 km shorter and pipeline section in permafrost will be shorter by 125 km. However, the route around the bay has the advantage of being adjacent to the railway for a long distance.

In addition, Russia has done the pre-feasibility study for developing the Shtokman gas field on the Barents's sea shelf, 600 km to the northeast of Murmansk. The sea depth varies from 280 to 320 m. Figure 1.3 shows possible alignments of the pipeline from Shtokman gas field to Teriberka, the closest point onshore, and to Rybachiy peninsula located west of Murmansk. Pipe diameters of 720 or 820 or 1020 mm are being considered. Construction from a pipe-laying vessel is feasible for the full length of the route, with the exception of the near-shore shallow water portion.

Russia has also studied two possibilities for gas pipelines from the Sakha Republic. The first option is from Kyzyl-Takhskoe gas field, west of Yakutsk, to Tynda and to Blagoveschensk. A further 200 km pipeline will be required to connect other gas fields in the Viluysko - Lenskoy oil and gas province with this main pipeline. Therefore, the total length of the pipeline will be 2550 km, or 2850 km if it is extended to Khabarovsk.

The second route is from the Kyzyl-Takhskoe gas field to Yakutsk to Tynda to Blagoveschensk. An additional 950 km of pipeline will be needed to connect the main pipeline with the Nensko - Botuobinskaya and Vysakhtakhskaya groups of gas fields. The total length of the pipeline is 3050 km. Both options have advantages and drawbacks. However, the proximity of the second route to roads and railways as well as the availability of an existing electrical power system and developed industrial infrastructure was decisive. In spite of the fact, that the second route is longer and correspondingly the pipe requirements is higher, it is estimated as preferable. The capacity of the gas pipeline is estimated up to 23 billion m^3/year, with approximately 50 percent for both domestic consumption and export. Options of 1220 mm and 1420 mm pipelines were studied. With a proposed operating pressure of 7.5 MPa, it is necessary to build 10 compressor stations and the same number of chilling gas stations. Gas would be chilled to -2 to -4°C. The capacity of the gas chilling station should be 20 MW and 30 MW for the pipelines of 1220 mm and 1420 mm respectively.

1.2.1.2 Main Design Features of the Russian Gas Pipelines in Permafrost

The design features will be discussed for the following main pipelines: Taas-Tumus - Yakutsk; Messoyakha - Norilsk; West Siberia - European part of Russia.

Gas pipeline Taas-Tumus - Yakutsk is laid in continuous permafrost, comprised of alluvial gravelly and coarse sands and lacustrine silty clays and clays. The mean annual permafrost temperature is -2 to -3°C. The thickness of the active layer varies from 0.5 to 0.6 m in mossy areas to 1.4 to 1.6 m in well drained, tree covered terrain. There are many swampy (wet) areas.

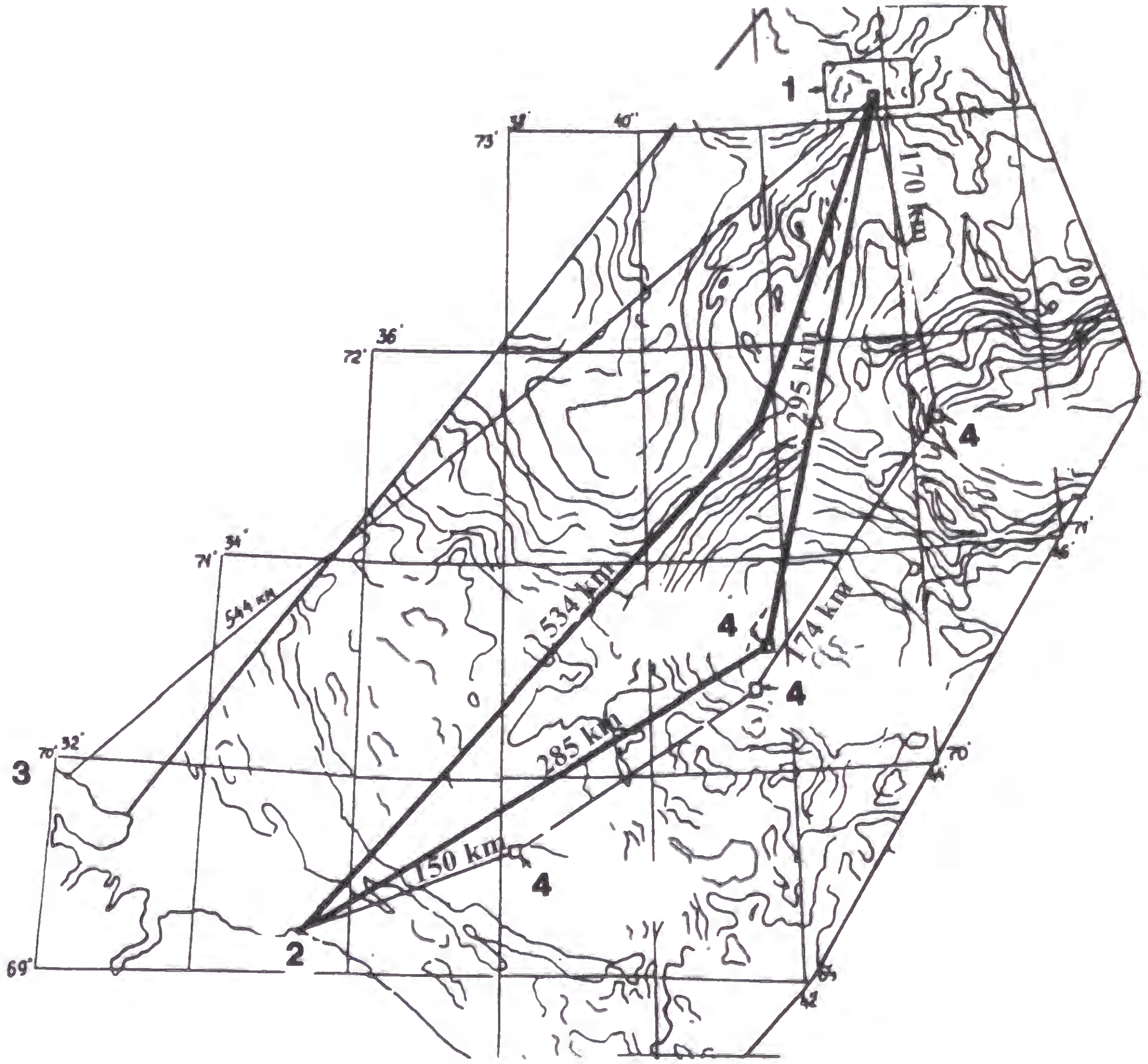


Figure 1.3: Versions of pipeline routes from Shtokman gas-condensate field
(1-Shtokman field, 2-Teriberka, 3-Rybachiy peninsula, 4-intermediate compressor stations)

Two construction modes were applied: buried, and above grade, placed within a pad. The trench depth was 1 to 1.5 m; the thickness of the pad is up to 3 m and the width at the base is up to 5 m. The thickness of the soil cover for the latter mode consists approximately 0.5 m over the pipe. The first mode was applied at terrains with relatively low ice content (less than 0.1). The second one was used at the terrains with high ice content (0.1 - 0.4). In swampy areas, the pipe was laid on the surface and covered with a mound of soil. Local soils consisting of silty clay and sandy silt were used for the pad and the covering. The right of way varies from 30 to 50 m. A bitumen-rubber mastic, 3 mm thick and a tape wrap were used for mechanical and moisture protection.

The gas pipeline Messoyakha - Norilsk. The route crosses 85 rivers, 15 large swamps and many lakes. The largest river is the Enisey, which has a width of 2500 m and depth up to 45 m. The rivers B. Khetta and Norilka have a width up to 500 m and depth up to 2 m. The route is characterized by continuous permafrost. The watersheds and slopes have mean annual permafrost temperatures of -4.0 to -6.5°C; flood plains have mean annual permafrost temperature -0.5 -1.0°C. Soils comprise clays and sands, with a little gravel. Somewhere, the flood plains have peat with thickness of 3 - 6 m. The frozen soils have high ice content (0.3 - 0.4); moreover, there are a lot of ice wedges. The mode of pipe laying for the field gathering pipelines and main pipeline is above grade on supports. The supports include, pile supports, timber crib supports filled with soil and earth supports with concrete slabs or ledges on top. The piles included concrete, timber and metal piles. The spacing between supports was 35 m for the first string; the spacing was shortened to 15 m for the second and third strings. The pipeline is elevated above the surface by 0.5 to 2.0 m. Crossings of small rivers and creeks was by the above surface mode.

The crossings through Enisey, B. Khetta, Norilka rivers was by buried mode, with each main string split into 2 or 3 strings with diameter of 529, 426 or 325 mm. The depth of the trench varies from 1 to 5 m. 2 or 3 pipes are laid in each trench with a spacing of about 5 m. The weighting for the pipeline is achieved by pig- iron weights.

Gas Pipelines West Siberia - European Part of Russia. The pipelines from the Yamburgskoe gas field to the Medvezhye gas field and the Urengoyskoe gas field are laid in continuous permafrost; the other pipelines are laid in discontinuous permafrost. The permafrost distribution varies from almost 100% (at Yamburgskoe gas field) to 20% (at Ob crossing near Peregrebnoe and Octyabrskoe) and 5% (at Ob crossing near Surgut). Forests cover 40 to 80% of the routes. West Siberia is characterized by a flat, poorly drained relief with many swamps and peat. As a rule, frozen soils can be encountered in peat, in clayey soils in wooded areas and in swampy wooded areas. The mean annual permafrost temperature varies from -2 to -3°C (Yamburgskoe, Medvezhye, Urengoyskoe gas fields) to 0 to -1.5°C (near the southern limit of all permafrost in West Siberia). The minimum thickness of the active layer is observed in peat (0.4 - 0.8 m); the maximum thickness is encountered in sands (up to 2.5 m). Silty clays have an active layer thickness of 1.0 to 1.8 m. The upper part of the frozen soils (to depths of 3 to 5 m) has relatively high total ice content: sands up to 0.2 to 0.35; sandy silts up to 0.25 to 0.4; silty clays and clays up to 0.45 to 0.55; peat up to 0.7 to 0.8.

Medvezhye gas field has 9 gas process units; Urengoyskoe and Yamburgskoe gas fields have 19 and 10 process units, respectively. The average number of wells in a cluster is 4, 5 and 10, respectively, for Medvezhye, Urengoyskoe and Yamburgskoe gas fields. The mode of pipe laying between the process units is on the surface with a soil cover. The

modes of pipe laying for the main transmission pipelines are different depending on a complexity of engineering geological conditions. The main modes of the laying of the gas pipelines West Siberia - European Part of Russia are presented in Table 1.1.

The table shows that for frozen soils with a thaw settlement more than 0.1 the above grade and surface modes of pipe laying were recommended. In practice, the above grade mode was not used owing to high cost and technical reasons. Beginning from the 1980's the surface mode is applied rarely. For example, the mode of the laying of the main pipeline Yamburgskoe gas field - Medvezhye gas field is buried. Polystyrene insulation of thickness 0.1 m was applied to prevent the surplus thawing under a pipe at the terrains with high ice content. A bitumen mastic and a couple of layers of tape were used for mechanical and hydro protection of the pipe.

Above surface mode was used for crossing through small rivers and creeks. The crossings through large rivers (Ob, Nadym) were done by multiple smaller diameter pipes using a manifold system. The pipe diameter in crossings is 720 mm or 1020 mm. The multiple pipes and manifold system is intended to serve the purpose of safety.

The trench for the overland buried mode has a depth of 1.5 m; the width of the trench base is around 2 m; the width of the trench near a surface equals 3 - 3.5 m. Russian and Japanese backhoes were used for the trenching. Before excavating, frozen soils are loosened by special bulldozers. The weighting of buried pipe in swampy areas was achieved by concrete weights, at spacing between 1 - 2 m. The weights comprise two half cylinders and are bound to the pipe by metal yokes.

TABLE 1.1 PIPE LAYING MODES - GAS PIPELINES FROM WEST SIBERIA TO EUROPEAN PART OF RUSSIA

COMPLEXITY OF ENGINEERING GEOLOGICAL CONDITIONS	TERRAIN	THAW SETTLEMENT	LAYING MODE		
			HOT PIPELINE	WARM PIPELINE	COLD PIPELINE
Medium	Terraces, Wooded Flood Plains	Less than 0.03	buried and on surface with soil cover	buried and on surface with soil cover	buried and on surface with soil cover
	Swamps		buried with weighting or within surface pad	buried with weighting or within surface pad	buried with weighting or within surface pad
Complex	Drained Tundras	0.03 - 0.1	above grade on pile supports or on surface supports	on surface with insulation or above grade	buried or on surface of pad
Very Complex	Peat, Solifluction Slopes	0.1 - 0.4	above grade on pile supports	above grade on pile supports or on surface supports	on surface with insulation and soil cover

1.2.1.3 Performance Problems With Some Russian Pipelines

Details on some problems experienced with several Russian pipelines in permafrost regions are contained in some of the Russian references available.

Taas-Tumus - Yakutsk Pipeline Investigations were carried out in 1975 - 1976 (Turbina, 1980). The buried mode and the above grade, in-pad mode were inspected. The mean annual permafrost temperature beneath the right of way had increased by 0.4 to 2.0°C; the thickness of the active layer had increased by a factor of 1.2 to 1.7 times. Construction works had significantly changed the drainage conditions. In many areas the pad acts as a dam and the presence of stagnant water beside the pipe is typical, even on shallow slopes.

The general conclusion is, however, that the construction works provoked insignificant development of thermokarst and thermo-erosion along the pipeline. The intense development of cryogenic processes is observed only in areas where wedge ice is present. The location of the pipe was reported to be generally stable.

Messoyakha - Norilsk pipeline Field investigations were carried out during many years at the sections of above grade mode and the buried mode (Enisey river crossing). Investigations included the influence of solifluction, thermo-erosion and aufeising on the pipeline integrity.

Disturbance of the vegetation and the cutting of slopes can initiate solifluction (slow, seasonal slope movement). Field investigations have shown that the soil of the active layer moves down along a slope with an average speed of 4 - 5 cm/month. The moving soil had caused damage to power lines and roads (Spiridonov and Garagulya, 1974).

Investigators concluded that thermo-erosion is a major destructive process for pipelines on permafrost slopes (Zamolotchikova and Chushkina, 1977). The cause of the thermo-erosion is the disturbance of the vegetation by construction equipment on the slopes. In 1970, the depth of erosion had reached 3 - 4 m.

The sections of pipelines with aufeis generally experience intensive frost heaving. The influence of the ice leads to pipe heaving, damage of insulation, and to destruction and damage of pipe supports. Monitoring of frost heaving at the aufeis sites were conducted in 1982 - 1985 in the valley of the Schuchya river. Investigations have shown that frost heaving is more intensive at the bottom of the valley than on the slopes (Rivkin, 1988). Total displacement of the piles are reported to have reached 2.4 to 2.9 m, with an average annual rate between 0.2 and 0.3 m. In 1985, the length of pile remaining in the soil does not exceed 3 m. Investigators note a correlation between the rate of aufeis growing and the rate of pipe heaving. Other factors causing intensive frost heaving are increases in the thickness of the active layer, high moisture content and the shallow depth of pile installations (Spiridonov, 1988).

Analysis of damage of the above grade pipeline sections has shown that the main cause of cracking and destruction of the pipes is aeroelastic vibrations. A pipeline of diameter 720 mm with the spacing between supports of 30 to 35 m has a frequency of aeroelastic vibration 100 to 120 per minute. As mentioned before, the first string of the pipeline was designed with spacing between supports (piles) of 30- 35 m. The second and third strings were designed with spacing of 15 m (Babenko *et al*, 1968). The vibration does not exist

if the spacing is 15 m and the pipe rests on each support. In reality, however, there are clearances between the pipe and many of the support beams. Clearances occur due to heaving, settlement and temperature deformations. The vibration can start when the clearance is only a few millimetres. The operating data shows that 80% of the damages occurred owing to the appearance of fatigue cracks near beams. Many years of observations have shown that constructing the beams at the height of 0.3 - 0.5 m above the surface can practically eliminate the vibration.

Extreme "cascade" ruptures, affecting in excess of 20 km of pipe, have been reported for the above grade section of the pipeline. The worst of these cases is reported to have occurred in February 1979, following an extended period of extremely cold temperatures (Repalov and Kharionovskiy, 1994).

The most complicated part of the Messoyakha - Norilsk pipeline is the crossing through the valley of the Enisey river, where there is a transition from above grade to buried mode. The gas temperature almost equals the air temperature at the point where the pipeline goes underground. The thickness of the active layer beneath the pipe is 1.2 to 2.2 m, depending on the depth of burial. The refreezing of the active layer under the pipe causes displacement as a result of frost heaving. The frost heave strain under the pipe varies from 0.08 to 0.12 and does not seem to depend on the depth of burial. It was reported that this pipe heave problem exists for a distance of 300 to 400 m from the start of the buried mode.

There were numerous pipe damage locations in the Enisey valley (buried mode) during the first five years of operation (Kondratyev, 1988). All damages have occurred from November to February and practically all of them have been at transitions between frozen and unfrozen soils. Investigators from Moscow State University give the following explanation of the causes of the damage. The gas temperature is negative in winter. The active layer freezes completely by December and the pipe is then restrained by frozen soil. However, due to the cold pipe temperatures, the freezing continues throughout the whole winter where the pipe is buried in unfrozen soil. Thus the pipe, which is restrained in frozen soil zones, suffers more and more stress from the heaving soils. Damage occurs when the heaving stress is more than the strength of the pipe steel, however, it is noted that almost all ruptures occur at welded joints.

West Siberia - Europe pipeline VNIIST carried out field investigations in 1974 - 1976. Thermo-erosion, disturbance of surface drainage, settlement and destruction of the pad were reported (Spiridonov *et al*, 1979). Where the pipeline crosses wet areas, there is damage to the cover pad. The worst pipeline integrity is noted at the crossings through swampy ravines, where the pad is generally damaged, the pipe is exposed and has come to the surface if it is not weighted. It was reported that the pad construction was of bad quality, apparently because large frozen fragments of soil had been used. After thawing, this pad contained many sinkholes and areas of large settlement and depressions, being the result of melting snow and ice.

A stabilization of some the permafrost degradation processes was observed at some pipeline sections. For example, three sites which were extremely wet in 1974, had become well drained in 1976.

VNIIST made the following conclusions on results of the field investigations:

1. The buried mode with weighting used at the crossings through swamps should not be used. During the first few years, the pipe comes to surface because the amount of the weights was not enough or most of the weights have sunk to one side and damaged the insulation. These crossings would better be constructed by exposed mode.

2. The crossings of narrow swampy creeks should also be constructed by the above surface mode. The crossings with buried modes were in bad condition. It is difficult to weight and lay pipe on the bottom of a flooded trench. As a result, the pipe is only partly buried, it disturbs the surface drainage and the pipe insulation is damaged.

3. The crossing of swamps by the above grade in-pad mode is accompanied by the disruption of the surface drainage and erosion of the pad. VNIIST recommends to use the exposed mode in swamps without building a cover pad.

4. The buried pipe on slightly swampy terrains should be weighted enough and the pad should be well compacted.

Kharyaga to Usinsk pipeline Much has been written in the press about the major 1994 oil spills from the Kharyaga pipeline. In 1995 the World Bank conducted an assessment of the integrity of the pipeline (AGRA, 1995). The 720 mm diameter pipeline, which was constructed in 1975, had a history of "hundreds" of minor leaks since 1988. In 1993 and 1994 major leaks started to occur. In August 1994, there was an "avalanche" of major leaks, resulting in spills estimated by some to total 200,000 tonnes. The integrity assessment of the pipeline concluded that the main causes were internal and external corrosion. The internal corrosion was caused by highly aggressive saline produced water being pumped with the oil. Estimates of the water content were as high as 43% of total throughput. The external corrosion was due primarily to the fact that the tape-wrap was very poorly applied. There was no cathodic protection. The pipe was buried in many swamps which contained slightly acidic water. The pipe had risen to the surface and was exposed to the atmosphere in several swampy areas due to inadequate weighting.

1.2.2 North American Experience

The author has met many times with numerous persons involved in research and design of several major North American pipelines and Russian pipelines in permafrost regions. The author has visited and made a special study of the Norman Wells pipeline, an oil pipeline buried in permafrost in Northern Canada. In addition, many books and publications have also been reviewed (AINA).

1.2.2.1 Early Pipelines

One of the earliest pipeline systems developed within the arctic permafrost terrain was the Canol Pipeline, shown on Figure 1.4. This pipeline was intended to supply petroleum products from a small refinery located at Norman Wells, NWT, Canada to Fairbanks, Alaska and was undertaken as a war measures development. There was concern that the petroleum requirements to satisfy both military and domestic operations within Alaska by marine transport was or could be in jeopardy through foreign intrusion. The pipeline project was conceived and developed by the U.S. military with the assistance of a U.S. contractor, H.C. Price.

The project was to consist of approximately 1500 kilometers of small diameter pipe (mainly NPS4 - Nominal Pipe Size 4") running from Norman Wells southwest to intersect with a minor roadway running primarily northwest through Whitehorse and on to Fairbanks. The work commenced during 1942 and oil was first pumped in late 1993. The majority of the materials, equipment and supplies were barged up the Mackenzie River, with limited supplies being trucked by winter road. The land was cleared and the pipe laid on the ground surface. The pipeline ceased to function when war ended in 1945, and the facilities were fundamentally abandoned.

This system had some considerable interest to later day pipeliners involved in the design and planning of the proposed large diameter pipelines from the arctic. Of particular interest was the ground surface settlement that had occurred due to the construction activities within the permafrost zones. The settlements were not as significant as anticipated and one of the most respected geotechnical specialists accredited this to the inherent good judgement of the old time pipeliners to select the most appropriate routing to avoid construction difficulties created by terrain conditions. Accurate isotherms across the right of way were obtained as benchmarks in assessing the accuracy of the geothermal predictive modeling capability.

Visitations to the camp sites and shops provided an appreciation of how difficult conditions were during construction. The equipment was inadequate for permafrost, camp accommodation was provided by tents and the shops provided an insight to the self dependency that was required to undertake such work in remote areas at that time. Of interest, is that the pipe was virtually in the same condition as when delivered to the site. It was salvaged in the late 1970's and early 1980's for reuse and environmental cleanup.

A similar pipeline system, the Haines - Fairbanks Pipeline (Figure 1.4), was installed during the same period. This system was located along the right of way of the Alaskan Highway and the pipe was placed in the ditches wherever grades created difficulties. This system operated for sometime as a products pipeline.

Fig. 1.4: Major North American Arctic Pipeline Projects and Test Facilities

1.2.2.2 Alyeska Pipeline

The planning and design for this pipeline commenced during the late 1960's. This system became the reference point for almost all proposed arctic pipeline systems that followed. The design and construction difficulties and modifications that were experienced became well known and served as guidelines for others. The initial approach to the design and construction planning was based on southern norms and failed to recognize the very significant difficulties that could and would be experienced. The initial conceptual design was based on complete burial and the pipe was apparently ordered on this basis. The potential thaw settlements and related upsets due to ground surface disturbance was recognized ultimately and modifications were introduced to assure that suitable roads and pads were utilized for transportation and construction purposes.

The flowing crude temperature is warm and necessitated elevating the system above ground for most permafrost areas. Trenching tests were conducted and confirmed the inadequacy of existing equipment to handle permafrost. The drilling of holes to install the vertical support members for the above ground segments proved to be most difficult, slow and costly. Pipe coatings were installed and modified on three occasions. A thermal set epoxy was selected and installed initially. One of the coating plants was located at Prudhoe Bay to process pipe arriving through the arctic ocean. This application represented one of the most significant applications when performance credibility was limited. Difficulties and problems with the then coated pipe became readily apparent when bending tests were undertaken. The coating would spall and crack readily due to inclusions existing within the coating plant environment. This coating was left in place and a tape coating was applied over the epoxy. Problems occurred with this coating due ultimately the lack of quality control. The accepted coating consisted of a crossed- linked polyethylene tape. Apparently the costs for the successive coating applications increased significantly in every case.

The management had little or no exposure to remote sites and arctic conditions. Such conditions were common for the Canadian construction companies and personnel. At that time it was common for workers to remain at a job site without rotation for a full work season. It was reported that there was an excessive administrative control over materials and supplies that destroyed attitudes, created indifference and seriously damaged productivity.

After the system was operating, the State of Alaska sued Alyeska for damages for creating unjustifiable capital costs that impacted on the state royalty income. Canadian engineers that were involved as expert witnesses had the opportunity to review relevant files and did not disagree with the State's position. One of the interesting facts that became apparent was that the above ground mode cost three times that of the buried mode. Considerable savings could have been achieved if the original pipe was replaced by heavier walled pipe, in which case the spanned distance between the VSM's could be increased by approximately one third.

This system has operated reasonably well since commissioned and was instrumental in creating an element of caution for others to undertake much greater research in permafrost behavior and the development of suitable equipment for arctic conditions.

1.2.2.3 Other Major Pipeline Projects

A significant number of large diameter natural gas and crude oil pipeline systems have been proposed and taken to the point of preliminary design and regulatory application preparation or filing. The following identifies these projects and provides an outline of each and any significance relating it. The earliest of these projects created a major undertaking to develop baseline data relating to all aspects of the terrain conditions and the wildlife existing within the arctic and subarctic regions of both Canada and the State of Alaska. Most of the involved territory was terrain-typed from photo mosaics and confirmed by drilling. This provided the capability of identifying borrow locations, preferable routing corridors, facility locations and potential infrastructure requirements. It also assists in assessing potential construction difficulties and probable production rates.

Environmental data gathering included most types of mammals (caribou, moose, grizzly bear, arctic fox, muskox, etc) , birds (waterfowl, raptors, ptarmigan, others) and, fish (all species). This data included population densities, distribution, seasonal behavior, and migration routes.

Mackenzie Valley Oil Pipeline

This project addressed transporting crude oil from the western arctic to connecting pipelines in the south. It was initiated by Imperial Oil (Exxon) and a number of pipeline companies. This was the first study to be initiated within Canada and had a reasonably short existence and was abandoned when natural gas proponents arose.

Canadian Arctic Gas Pipeline

This identity was created by the merger of two competing groups proposing pipeline systems connecting Alaskan gas to existing pipeline systems to the south. These competing groups were Gas Arctic and the Northwest Project. After the merger, twenty seven sponsors were involved consisting of most major producers and natural gas pipeline companies within the USA and Canada. Tens of millions of dollars and many years were spent in the engineering, construction planning, environmental assessments, native orientation and training and regulatory applications and hearings. Nova Corporation disapproved with the lack of sensitivities to Canadian interests and involvements and withdrew from the project and proceeded to prepare a competing application with Westcoast Transmission, another Canadian Pipeline company. The later project was called the Maple Leaf Project and proposed a routing up the Mackenzie River basin to connect with production in the Western Canadian Arctic. After bitter fighting and prolonged hearings, the Maple Leaf project was awarded the permit to develop an alternative system that avoided the Mackenzie River Basin with the potential production from both the Alaskan and Canadian fields. This was a tremendous surprise because there was no supporting data or application submission for this alternative. The successful group abandoned the Maple Leaf identity and created the Foothills Pipeline Project in conjunction with the US Northwest Group.

Alaska Highway Gas (Foothills) Pipeline

The Foothills Project consisted of two components. The one consisted of a prebuild segment to provide an early start to the system by development of the southern portion to transport natural gas produced within Alberta. This segment was completed during the early 1980's.

The second portion addressed the segment covering the northern component of the system, basically following the Alaskan Highway and with the plan to connect to the Northwest System at the Alaskan border. For a variety of reasons the conceptual design had to be abandoned and many modifications introduced for cost effectiveness. The routing alignment contained a large component of intermittent permafrost and presented the alternatives of chilled gas flow to avoid the problem of differential settlement and address the possibility of frost heave problems. Alternately, warm flow would certainly create differential settlement problems.

The solution to eliminate both concerns within the intermittent permafrost zone was utilization of a gravel berm covering the pipe laid on the ground surface. Assessments of the berm concept indicated that the necessary quantity of borrow materials required created supply difficulties and excessive costs. The apparent solution was to retain the concept but replace the berm with a concrete cover over the pipe. Concrete blocks in the form of a saddle weight and positioned in an inverted position were designed and tested. These blocks were designed to provide the necessary protection while assuring constraint to any pipe movement. This concept would have provided tremendous savings compared to work required to source and position the materials required for a basic berm covering. The design development for this project was extensive and addressed all concerns that were recognized including such items as heat tracing road culverts. This northern segment of the project was never developed because of natural gas discoveries within the Canadian arctic that could be accessed more economically.

Polar Gas Project

This project was conceived during the late 1970's with the intent of connecting the gas discoveries on the Canadian arctic islands to markets within the eastern USA. Routing east of Hudson Bay was initially adopted for this project. This was abandoned for a number of reasons, including terrain difficulties and development resistance by the aboriginal people. Alternative routing to the west side of Hudson Bay was adopted and studied. New discoveries in the arctic islands and western arctic made the project to explore a bifurcated system to serve the two areas. Further production capability and potential in the western arctic provided the lower capital costs and a decision was made to focus only on the Mackenzie Valley with the potential of tying-in the high arctic production at a later date. Several of the sponsoring companies withdrew from the project, leaving only Trans Canada Pipeline and Tennaco. Results from continued drilling proved disappointing and the project was placed on hold. One unique aspect of this project was the requirement to construct marine pipelines to connect the islands. Field programs were undertaken to define subsurface conditions, currents and ice movements and potential damage to the lines. Bottom pull techniques were considered the most appropriate installation technique and the project examined techniques for underwater tie-ins and repair techniques. Fast-ice conditions along the shoreline have to addressed along with iceberg scour. The fast-ice does create potential scour conditions of a maximum depth approximating 3 metres during breakup.

Arctic Pilot Project

This project was initiated to address the potential transport of LNG's from Melville Island in the high arctic to markets in Europe and Eastern North America. Melville Island is located at the latitude maintaining year round ice conditions and at the best of conditions can only be reached by marine transportation during a one-month window. Planning consisted of a significant pipeline, barge-mounted LNG facilities and ice breaking tankers, along with significant assessment of ice behavior and conditions and water depths necessary to select the most suitable routes for the tankers. This project was processed through regional and full National Energy Board Hearings but never developed.

Beaufort Delta Oil Pipeline

This project addressed crude oil transportation from the Mackenzie River delta to southern Canada. It was sponsored by a group of oil majors, however, it folded after approximately one year. The shut down justification was not particularly evident but was likely tied to the economics based on oil pricing existing at that time.

Norman Wells Oil Pipeline

This project was conceived by Esso in association with the rework and the expansion of the Norman Wells oil field. Five artificial island were installed in the Mackenzie River and multiple direction well drilling from each island was undertaken. Production was piped to both natural islands (Goose and Bear) and then to the mainland. The main transmission pipeline was proposed from this subarctic location to existing pipelines to the south. It was decided that this 870 km line of NPS12 should be identified as a pilot project to avoid extensive regulatory hearings and possible native interference. It was also decided to attempt to route the line within existing cut lines servicing seismic activities and an abandoned telegraph line. Many deviations to routing were made to follow this principle but had minor influence on costs such as would occur on a large diameter system.

Esso proposed a two phase system transporting natural gas in solution with crude oil. More detailed evaluation of such a system showed no particular savings and many potential problems. The operating pressure had to be much higher than normal and to satisfy logical station spacing would have to be in the 13,000 kPa range. At stations the gas had to bypass the pump units and be injected again. The main concern was related to breaks within a two phase system. Any break would release an oily froth that could create significant environmental damage particularly if entered the river basin. Two lower pressure buried pipes within a common trench showed comparable costs. On this basis, it was decided to eliminate the two phase system and transport only crude oil.

The proposed pipeline received no difficulty in receiving regulatory approval and although a larger system was desired it was decided not to pursue this increase in the belief that is would indicate a lack of good faith by the proponents.

During the first summer of construction, wharves, stockpile sites, camps, infrastructure, connecting roads and borrow sites were developed. All clearing of timber had been

completed during the previous winter. This helps identify potential problems, provides contractors a better opportunity to assess grades, rock, "shoo flies" or bypass roads necessary for vehicular travel to avoid ravines and other impassible sites. This also allows the contractor the opportunity eliminate many of the perceived risks and typically results in reducing the bid prices.

Pipe and contractors equipment were barged to various sites within the northerly half of the pipeline during late summer and early fall. Freeze up of the Mackenzie river system normally occurs during late September. Some portions of the northern half of the system required delivery by truck. This necessitated the construction of an ice bridge over the Mackenzie River. Construction of this required approximately one month to assure suitability and the strength to handle heavy loads. Heavy trucks represent point loading of over 500 kPa squared compared to tracked equipment loading of less than 140 kPa. The planned work structure for six spread seasons over a two year period. Two contractors were awarded the pipelaying, Separate contractors were awarded for the Crossing of the Mackenzie River. and the stations.

Pipe laying operations commenced at the extreme ends of the system and excellent production was achieved. Production averaged almost 8 km per day for each spread using double pipe gangs. This unexpected rate resulted in the requirement of only five spread seasons of work. This project was completed well within schedule and at a cost approximately fifteen percent below budget.

It is understood that the key or unique elements of this project are the following:

- graded slopes were protected by wood chips obtained at site to minimize the thermal effects of construction disturbance on the permafrost.
- flowing oil temperatures were designed to remain within one degree of the soils temperature and appeared to be achieved.
- aboriginal peoples were awarded an unusually high proportion of the work and performed it successfully.

1.2.2.4 Research Test Facilities

The above projects undertook a great quantity of field testing to provide design criteria and understandings and to assure the strength and productivity of equipment. The following section outlines the test sites, their objectives and some of the results. These sites range from large installations to a wide diversity of field testing of equipment, materials, and construction techniques.

Prudhoe Bay Test Site

Sponsorship: Gas Arctic Project sponsored by Nova, Texas Eastern, Columbia Gas and Northern Natural Gas.

Description: The facilities consisted of a large diameter pipe loop with three ditch burial alternatives; full burial, neutral or part burial with a berm cover and sole cover by a berm. Air was circulated and the temperature could be modified to a limited degree. In addition, a deep large diameter vertical shaft was installed to accommodate visitors and staff to observe the subsurface permafrost distribution. A snow/ice road installation was included to assess the capability to withstand high volume vehicular traffic. To some extent this site was developed for political aspects, namely to impress regulators and government scientists of the intent to support engineering with research backup. The cost was significant and greatly exceeded the costs for facilities developed solely for obtaining technical data.

Test Results:

- excavation at the site required blasting of the ditch line. This provided the insight that blasting control within permafrost cannot be controlled in the same manner as in rock. It certainly made excavation easy but left micro fissures extending a considerable distance from the ditch wall.

- caribou grazed consistently within the center of the test facility. The site had been seeded and produced a good crop of grass. Geese nested on the berms and both along with smaller mammals showed little concern for the noise from the air compression and the near proximity of humans. This was the opposite to the advice and forecast of the environmental scientists.

- the site was visited by a wide range of project executives, staff from the government agencies and personnel involved in the Alyeska Project

- the snow roads stood up well and ground vegetation received little damage except for the fact that most did not flower during the following year.

- ice rich soils stockpiled for repair of berms and other site requirements, showed a high degree of desiccation and would basically powder when disturbed.

- snow distribution and snow collection programs were undertaken in conjunction with this. A snow fence was installed to determine the time duration required cover the fence. To everyone's surprise this took approximately fifteen minutes instead of the one month period that had been recommended by personnel conducting this work. This is not uncommon in tundra regions and was later used for other testing purposes. The constant drifting that occurs desiccates the snow and results in higher densities and packing capability than occur in the more sheltered boreal forest regions.

Norman Wells Test Site

Sponsorship: Gas Arctic

Description: This facility was a low budget installation developed to gather scientific data and test construction related concerns. Meetings were held with the National Research Council of Canada to obtain their input on how the test facility could best serve industry's knowledge and requirements relating to permafrost. Their recommendations focused on the need of hard data relating to geothermal properties and particularly identified heat flux data and accurate measurement of isotherm changes created by some degree of controlled heat. It was considered that such data was the only way to obtain confidence in the predictive modeling.

The facility was relatively simple with two segments of pipe consisting of a pipe within pipe to achieve circulation of warm air to simulate flowing gas temperatures. The test pipe segments were designed to handle a different range of temperatures. There was a third segment installed with capped ends and no air circulation. Its' sole purpose was to check the buoyancy influence created by thawing of the active layer during the summer period. There was a concern that floatation could occur in a range of soils. The site was heavily instrumented to obtain thermal data. The site was also utilized to test snow/ice roads in the boreal forest region and a simple ditching test was conducted to check the capability of a large backhoe with a three yard bucket to excavate without blasting assistance.

Of interest was the fact that a heat exchanger installed on a Caterpillar Generator unit provided sufficient heat satisfy the heating of two buildings and the two test loops.

Test Results:

A very considerable amount of geothermal data was accumulated and used to fine tune modeling capability representing conditions during all seasons.

- snow road testing proved that either ice or snow bases could be utilized successfully but either would require a reasonable amount of maintenance.

- the large sized hoe was unable to penetrate the permafrost soils and it was a true demonstration of potential challenges.

- the pipe section that was installed to check buoyancy did not move despite the fact the soils were water saturated.

- testing of the possible use of rod anchoring was undertaken using reinforcing rods positioned within drill holes and restrained by an add freeze mixture of wet soils and water alone. These rods demonstrated tremendous strength initially, to the point breaking the 3/4" rods. With time, however, creeping was evidenced and the approach was not pursued. This could serve as an inexpensive method of anchoring during a limited period.

Inuvik Test Site

Sponsorship: Mackenzie Valley Project

Description: This was the earliest test facility installed in Canada. It was basically a pipe loop consisting mainly of buried pipe and a much shorter segment of above ground pipe circulating hot oil.

Test Results:

The testing program was short lived and no technical data appeared to be gathered. Obviously rather deep subsidence of the ground surface occurred but the testing appeared to confirm the fact that burial of hot oil lines in ice rich terrain could not be successfully be considered.

Sans Sous Rapids Test Site

Sponsorship: Northern Pipeline Project

Description: This installation was installed on the west of the Mackenzie River north of the Village of Norman Wells. The location was inaccessible except by helicopter or float plane or boat. This limited visitors to the site to project personnel or guests invited by the sponsors. It consisted of intermediate sized pipe loops circulating air. Published information was limited and little information relating to the testing and results was made public. This is surprising in the facility was obviously costly. This project was merged with Gas Arctic to form Arctic Gas. After the merger, little or none of the results were included in the final design phase and it was assumed that the program provided little technical data compared to other sites.

Quill Creek Test Site

Sponsorship: Foothills Pipelines (Yukon) Limited.

Description: This test site was developed in 1981 for the Alaska Highway Gas Pipeline Project. The test site incorporated numerous components, from buried and above-ground pipeline construction modes; snow/ice road construction and trafficability; ice-rich slope performance with various forms of mitigation. Figures 1.5 and 1.6 illustrate some of the main components of this test site. The main design mode developed and tested for this project, was the above-ground, concrete restrained mode (Figure 1.6). Some of the terrain was very ice rich and the hot gas pipeline would have cause excessive thaw settlement. An above-ground mode was required, however, an alternative was sought compared to the very expensive vertical support members (VSM) as used for Alyeska. The basic concept was to place the pipe on the surface, with a layer of insulation beneath the pipe and then to cover the pipe for security and restraint against thermal expansion forces. The desirable soil cover would consist of sand and gravel, however, in many sections of the pipeline, there was a very limited supply of natural sand and gravel. Hence the idea of using specially formed, insulated concrete weights as shown on Figure 1.6.

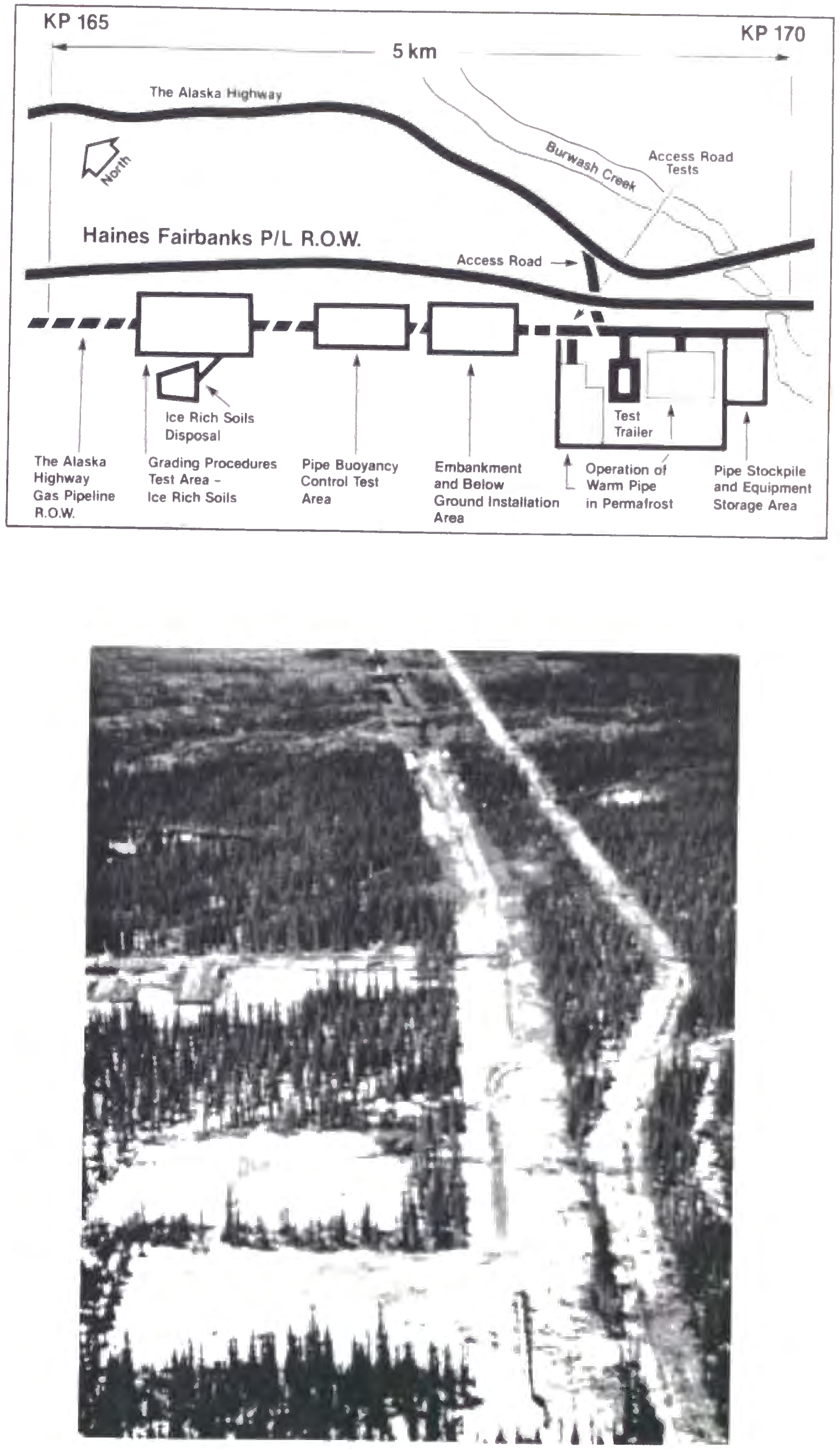


Figure 1.5: Layout of Quill Creek test site (Fielder, 1986)

Figure 1.6: Quill Creek test site - concrete restrained mode (Fielder, 1986)

Test Results:

The conclusions from the test facility further confirmed that large diameter pipelines could be installed in areas containing ice-rich permafrost without causing unacceptable damage to the environment. The installation of insulated pipe both above and below ground has been proven feasible. The construction and performance of the unique, concrete restrained mode were successful and the geothermal performance of all modes were close to predictions.

Ditching Tests

General:

Pipeline trenching within Canada during the winter season has historically created problems affecting productivity, particularly for large diameter pipelines. Up until the mid 1960's, most spreads utilized pilot trenching or ripping techniques along with three ditchers to achieve productivity comparable to welding capability. Usually the equipment required considerable maintenance as breakdowns were common and multiple shifts were the norm.

As pipe sizes being installed increased in diameter, more interest was attracted to this problem, and a number of contractors developed hybrid ditchers for their own utilization. Horsepower was increased substantially; hydraulic drives were introduced and drive segments were positioned on both sides of the wheel rims. Commercial designs utilized chain drives and outside segments to drive the wheel. This arrangement created chronic problems. Ditching teeth had limited life and required changing at the end of most shifts. Such modifications allowed wider buckets to be utilized. With these modifications, productivity increased tremendously to the point that a single ditcher could satisfy target productivity. New teeth design and metallurgical components provided improved performance and reduced the requirement teeth changes.

Alyeska tested one of these "jumbo" ditchers and proved that it was no match for permafrost. Canadian arctic planners recognized the importance of this limitation and commenced programs for new developments associated with trenching. The government even financed considerable funds for these programs. The major problem became ditcher tooth design to withstand the service requirements within permafrost regions. Many different configurations were developed and tested unsuccessfully. It became apparent that a single design would not satisfy the various soil types creating impact or abrasive damage to the teeth. The attack angles were modified and various metallurgical components were introduced. The last ditching test that was undertaken and it demonstrated the capability to achieve one mile per eight hour shift in the worst of conditions, excluding bedrock.

Norman Wells:

This test was undertaken reasonably early in the ditcher development programs. The ditch site was located south of Norman Wells, within a frozen gravel deposit. This test was considered reasonably successful but did only limited capability to perform to the necessary standards. One of the major problems involved teeth strength under impact loading created by boulders and gravel.

Churchill:

This site was located on the west side of Hudsons Bay. It was chosen as it could be accessed by rail and allowed the large trenchers to be transported to a permafrost site after freeze up. This region has a broad distribution of permafrost and contains cobbles within the selected test area. The test was undertaken to compare the relative performance of two arctic trenchers. The largest was newly developed and incorporated some unique design approaches.

The boulders (cobbles) created a number of problems particularly with the teeth. The new unit performed much better than the other machine that had years of development. This created some satisfaction but also the realization that more development was required.

Melville Island:

This test site was in the high arctic, and represented the most severe terrain conditions likely to be encountered. Two different soils types were selected to represent both abrasive (sandstone) and impact conditions (bouldery till) for the ditcher teeth. There is a negligible active layer at this site and the ground temperatures are much colder than further south. The test objectives were to assess the machine and teeth under these conditions using six foot wide buckets. With high quality metallurgical teeth, new attack angles and modified configurations of the teeth position around the bucket face, it was hoped that teeth life could be extended to a minimum duration of eight hours. This represents a single shift and the changeover for crews. The tests confirmed the capability to satisfy the targeted objectives. During the shift of eight hours, a minimum production of one mile was achieved. It was the opinion of the construction experts involved, that excellent production would be achieved in less challenging terrain and weather conditions.

Ditcher Testing Conclusions:

The testing of ditching techniques for arctic conditions has had a significant impact on the design of commercial trencher design. Many of the concepts developed for the arctic have integrated into commercial equipment and the large trenchers are utilized commonly on both winter and summer work because of their productivity and ease in maintenance.

Norman Wells Pipeline Ditching Production

The arctic ditchers successfully developed for the larger diameter pipeline projects, were used for the more northerly spreads on the Norman Wells project. These units were twin-engine, 1200 hp wheel ditchers. The wheels and buckets were modified for the smaller ditch dimensions (0.7 m wide, 1.1 m deep). These large ditchers were very efficient in most terrain conditions, however, production was considerably slower in a localized, very bouldery terrain. Figure 1.7 shows

the rate of progress made during the first winter construction season in the most northerly spread. This progress was accomplished by two arctic ditchers, for the most part (one ditcher broke through the frozen crust in a wet swampy area and was out of commission from about Day 36). These production rates varied from an average of about 1.8 km per unit per day, to a peak production of at least 6 km per unit per day.

Snow Road Testing:

Trafficability testing was undertaken at Prudhoe Bay, Norman Wells and Inuvik. The construction of such roadways was very much influenced by US Military procedures developed for airstrips and roadways considerably before the 1970's.

These procedures covered positioning, packing, scintering, and repacking of the snow bed. Scintering a beating of the snow similar to cultivation of fields. This process is necessary strengthen the snow and creating a greater density to the roadway.

The test program at Inuvik was scheduled in early December. The snowfall at that time was minuscule and insufficient for the Testing. The necessary equipment was located at site and It was decided to collect snow from a nearby lake and haul to the test site. The accumulation proceeded rapidly and loading and hauling commenced shortly after began. The trucks were end dumping the snow on to the proposed alignment. The density of the snow was sufficient to allow the trucks to travel over the unpacked snow without difficulty. This allowed much greater flexibility in modifying the depth of fill and consequently resulted in better grades both on the centerline and on bends. The road required only minor maintenance and the test proved to be very successful. Additional testing involving the use of snow making system from a ski hill. The modification of the water - air mixture allows a slush to be produced. This capability was considered extremely advantages for the grade filling of gullies or ditches without the need of snow hauling or problems where the depth of fill could not satisfy the density requirements.

The current knowledge and capabilities are sufficiently mature that little or no development would be required to utilize them on any new arctic applications.

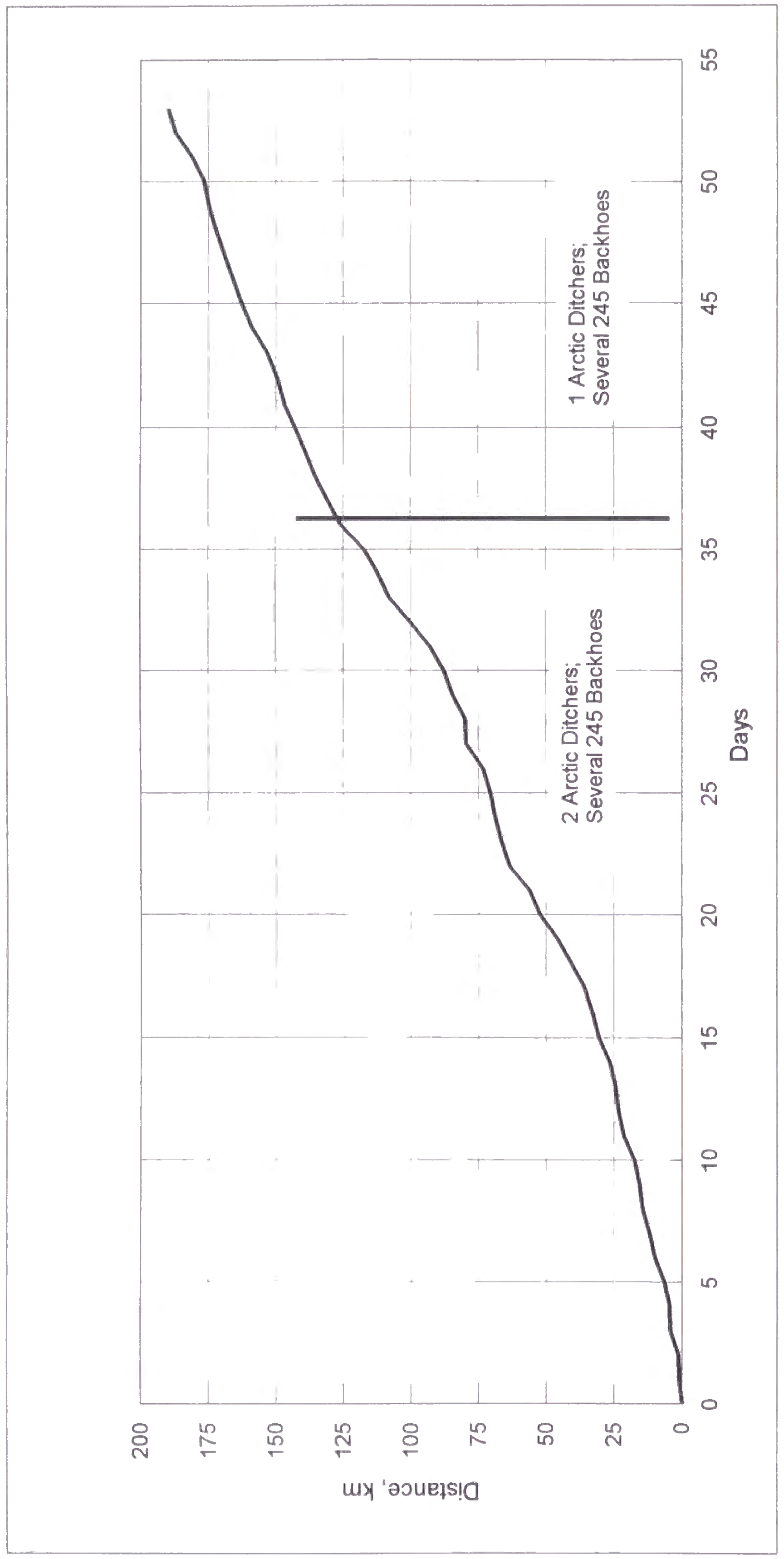


Figure 1.7: Norman Wells pipeline ditching progress (winter 1984)

Ice Heave Testing:

This phenomena was recognized during the early planning and design approaches to arctic pipeline transporting natural gas. Laboratory testing was undertaken at the Universities of Calgary and Fairbanks to attempt to quantify the extent of the potential problems. Frost heave requires combination of a number of conditions. These are a cold temperature environment, free water and fine soils such as silt. Water would gradually migrate to coldest temperature source, causing ice lens buildup under an installed pipe. This ice is considered to have the strength to deform the pipe.

No particular concern was shown by designers until an intervenor at a regulatory hearing challenged the lack of concern and presented rational arguments based on his theories supporting the occurrence of frost heave and it's potential damage to pipelines . When no strong arguments by the application experts, countering his theories and hypotheses, it literally destroyed the credibility of the proponent and much of the valuable research that was undertaken and the permit was not issued. No evidence has come forward to indicate that actual damage to existing pipeline systems has occurred. Testing was continued at several locations but the results are not generally Known.

At the Norman Wells hearings the potential for crude oil pipelines operating at a temperature close to freezing was introduced This could only occur within the intermittent permafrost zone and where operating temperatures fall below freezing. If this condition exists in such locations under water courses, frost heave could possibly occur. To protect against the possibility of permit denial, the proponent undertook to participation of an intelligent pig with the capability of accurately measuring pipe movements and potential damage due to frost heave. Such tools have been developed that can measure not only movement but geometry of the pipe and corrosion damage. Since this development, some advocates have arisen to challenge the concept of the potentially perfect pipeline that can satisfy all conditions. This does not occur on normal pipeline system developments and there is little justification for special conditions or requirements applying only to arctic development. These advocates believe that considerable investment could be saved or delayed by routine monitoring of such pipeline systems and only address frost heave and other such problems only after evidence of such problems are identified.

Pipe Burst Testing:

Their first testing was undertaken by Arctic Gas Project. The standard rolling techniques to achieve steel strength throughout the total wall thicknesses being selected. To assess the strengths that were achieved, testing was undertaken to determine the actual strengths that were obtained. This test was conducted at the test site of the American Gas Association, under the direction of metallurgical experts from the Batelle Institute. Normal testing structures and procedures were applied in the belief that fracture arrest would occur within the pipe test segment. When the fracture was initiated, it ran through a large reinforced concrete anchor block and sent debris far beyond normal distances. Some observers reported that some of their peers refused to examine the pipe due to the fear generated by the magnitude of the explosion. This occurrence was the first exposure that many industry personnel had to ductile failure. This occurs due to Joule Thompson effects that produce cooling due to gas pressure reduction. In the case of large diameter, high pressure pipelines, the release of gas is so profound that freezing of the fracture tip occurs in conjunction with the fracture. The strength of the pipe is reduced by the freezing and the fracture can run unabated until it reaches an appurtenance of some strength such as a valve installation. The fracture runs in a straight line along the upper side of the pipe and in the process hurls debris over a large area. On such systems it is now common practice to install fracture arresters at reasonably close intervals along the pipe. Such arresters can consist of lengths of heavy wall pipe, sleeves or cable loops around the pipe.

Burst tests were conducted by the Foothills Project at a site in Northern Alberta. the results of these tests were not disclosed.

2.0 JAPAN'S ENERGY POLICIES AND NEEDS FOR NATURAL GAS

Eight representative companies in Japanese industry (Nippon Steel, NKK, Tokyo Electric Power, Tokyo Gas, Osaka Gas, Mitsubishi Heavy Industries, Mitsubishi Research Institute and Industrial Bank of Japan) established the "National Pipeline Research Society of Japan" in 1989. Prof. Hirata (then of Mechanical Information Engineering at the Department of Technology, Tokyo University) was appointed the chair for research and investigation of the actual conditions and concepts for the improvement of large diameter natural gas pipelines systems within Japan and abroad. The author has since been leading and promoting these activities as an executive of the society. At present this society has about 50 member companies representing Japan.

During the seven-year period from the inauguration of the society to the present, the author made more than ten field trips to oil and natural gas producing regions and pipeline projects with Prof. Hirata and conducted interviews with high government officials of various countries. These interviews and visitations provided the opportunity to become aware of the considerable geopolitical and other problems associated with the development of major multinational energy projects.

The author also realized that in order to promote the construction of a natural gas pipeline within Japan for the 21st century, it is necessary to undertake in parallel, the study of alternative pipeline systems required for the importation of foreign natural gas.

Based on this realization the author recently published, with Prof. Hirata, the "Conception of an Asian and Pacific Energy Community" which addresses particularly the concept of constructing a pipeline network, as shown on Figure 2.1, connecting natural gas fields in the Asian area, called the "Proposal on The Trans-Asian Natural Gas Pipeline Project" (National Pipeline Research Society of Japan, 1993). Later in 1993, former Prime Minister Keating of Australia said that such a pipeline network project was the "only" theme suitable for discussion in the framework of the APEC and that it would serve to cool down the tensions among the Asian and Pacific countries which were still continuing their arms buildup.

In March 1995, the first "Northeast Asian Natural Gas Pipeline Symposium" was held in Tokyo. The second was held in Beijing in September 1996, with a large attendance of people from various countries. At this meeting it was proposed and decided that future symposiums be held annually in Japan, ROK, China, Russia, etc. on a rotational basis under the new name of "Northeast Asian Natural Gas Symposium". The Japanese government is providing more support for the symposium, mainly by the Ministry of International Trade and Industry (MITI) and the Ministry of Foreign Affairs (MFA), and the government arranged to have retired, high-ranking officials of MITI participate in the second symposium in Beijing as panelists.

Why then has the necessity of constructing natural gas pipelines become such an important subject of discussion? To answer this question it is first necessary to review Japanese energy policies. According to an interim report (June 1995) of the Supply-Demand Section of the Advisory Committee for Energy, an advisory organ of MITI, Japan's primary energy supply forecast is set as shown in Table 2.1.

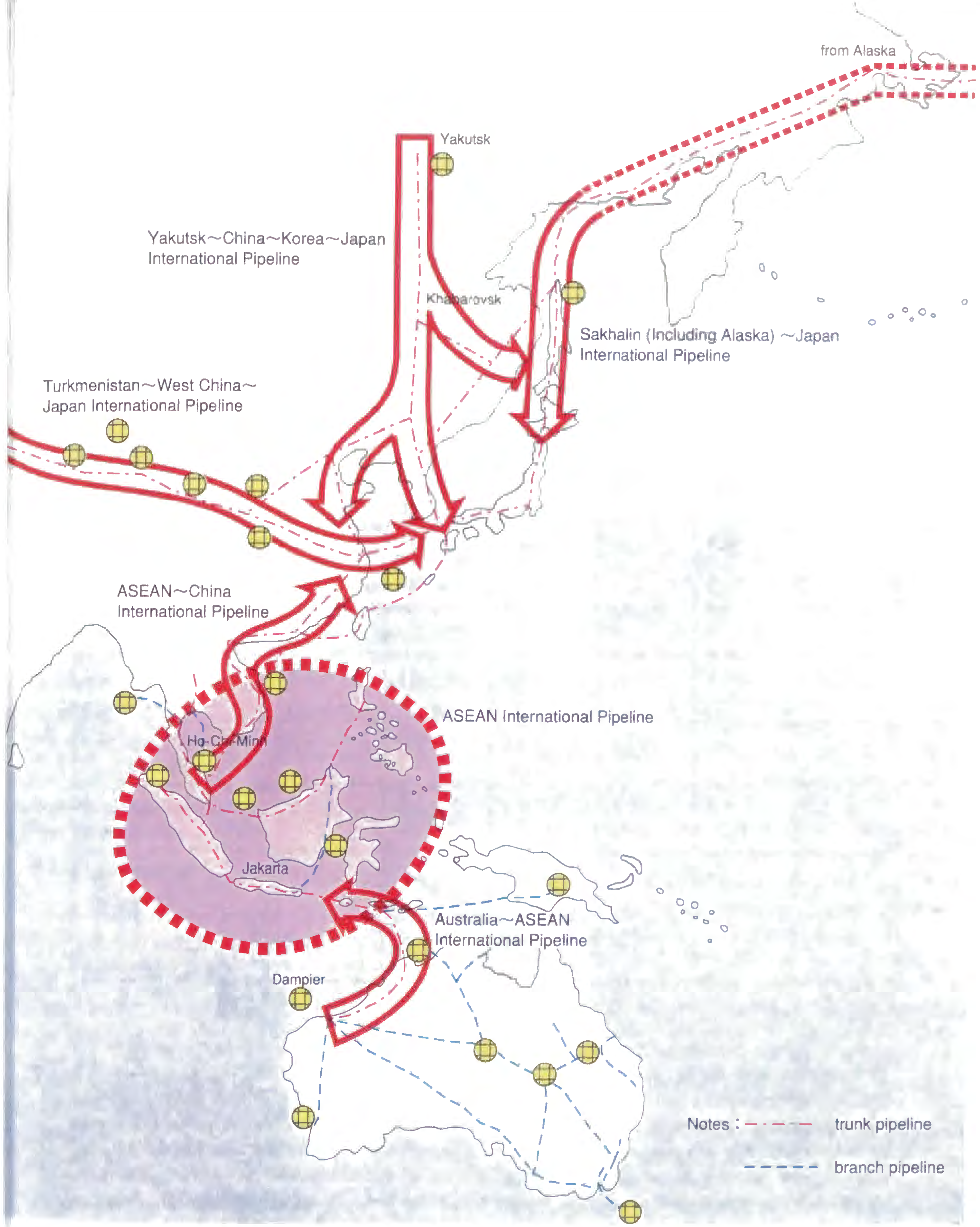


Figure 2.1: Proposed Trans-Asian natural gas pipeline project

(National Pipeline Research Society of Japan, 1993)

Of the values in the table, the nuclear energy increase rate is the most serious and important problem. The expected supply of nuclear energy is 45.6 million kW in 2000 and 70.5 million kW in 2010 showing an increase of 24.9 million kW during the decade. As the output of one nuclear power plant is about 1.0 million kW, this would necessitate the construction of an additional 25 plants. However, it would be very difficult to construct 25 plants in that decade because the lead time for the construction of one nuclear power plant in Japan at present is said to be 26 years. As of December 1995 Japan has 50 nuclear power plants in operation, four under construction and two under contemplation. According to MITI's long-range forecast, Japan would need to construct another 50 new nuclear power plants by the Year 2030.

Table 2.1: Japan's Primary Energy Supply Forecast

Fiscal Year / Item	1992 (actual)		2000		2010	
Oil (million kl)	314	58.2%	316	53.4%	331	50.1%
Coal (million t)	116.3	16.1%	134	16.6%	140	15.3%
Natural Gas (million t)	40.7	10.6%	54	12.8%	60	12.7%
Nuclear Energy (million kW)	24.4	10.0%	45.6	12.1%	70.5	12.7%
Hydraulic Energy (million kW)	21	3.8%	22.2	3.3%	26.5	3.5%
Geothermal Energy (million kl)	0.55	0.1%	1.0	0.2%	3.8	0.6%
New Energy (million kl)	6.7	1.2%	9.4	1.6%	11.5	1.7%
Total (million kl)	541	100%	591	100%	662	100%

It should be noted that in the resident's poll of August 1996 regarding the construction of a nuclear power plant at Makimachi, Niigata Prefecture, the opposition gained a majority position. This was due to the fact that measures taken by electric power companies and power resources developers to protect against previous nuclear accidents were considered inadequate and also because the resident have an increased awareness of the environmental problems. This fact is considered to pose a critical test for future nuclear power plant projects. If that is the case, how could the power shortage, resulting from delayed or canceled construction of some of the 23 or more nuclear plants be replaced?

Consumption of coal and oil as alternative energy sources cannot be increased easily because of the worldwide environmental concerns. Wind power, solar thermal power, biothermal power etc., as new energy sources are so small in quantity that they cannot cover the shortage either.

Therefore, what remains is only natural gas, for which there is a growing demand as it has been recognized throughout the world as a clean source of energy.

Japan presently imports gas in the form of LNG (liquefied natural gas) from the countries shown in Table 2.2. As of 1996 Japan was the world's largest LNG importer. Of the total imports of 47.2 million tons, 75 percent, or 35.4 million tons, are consumed by electric power companies, about 28 percent by city gas companies and the remaining amount by Nippon Steel, as fuel for power generation by cooperative power plants.

Table 2.2: Japan's Actual Imports of LNG (fiscal 1996)

No.	Country	Import (million t)	Component Ratio (%)
1	Indonesia	18.36	38.9
2	Malaysia	9.22	19.5
3	Brunei	5.55	11.8
4	Australia	7.17	15.2
5	Abu-Dhabi	4.30	9.1
6	Alaska	1.30	2.8
7	Qatar	1.29	2.7
	Total	47.19	100.0

Most of Japanese electric power companies and city gas companies, which have so far been importing LNG, have long-term contracts of 20 years or more. Therefore, it seems that there is no concern up to about 2010 about the supply quantity established by MITI's primary energy supply forecast shown in Table 2.1. However, MITI's demand forecasts in 2000 and 2010 are much lower than those of our "National Pipeline Research Society of Japan" (see Figure 2.2). Even ignoring the difference of 35 million tons/year between the MITI and the Society forecast values for 2010, an increase of 23 million tons/year of LNG will have to be secured over this decade, if the nuclear power plant projects are not developed.

Indonesia is the world's largest LNG producer and exporter. The annual export quantity reaches 24 million tons, of which 70 percent or more is imported by Japan. The LNG import contract of 1973, which would have expired in 1999, has recently been extended for 11 years to the end of 2010 and the contract of 1983, which would have expired in 2003, has also been extended for eight years to the end of March 2011, but some price increases seem to be inevitable.

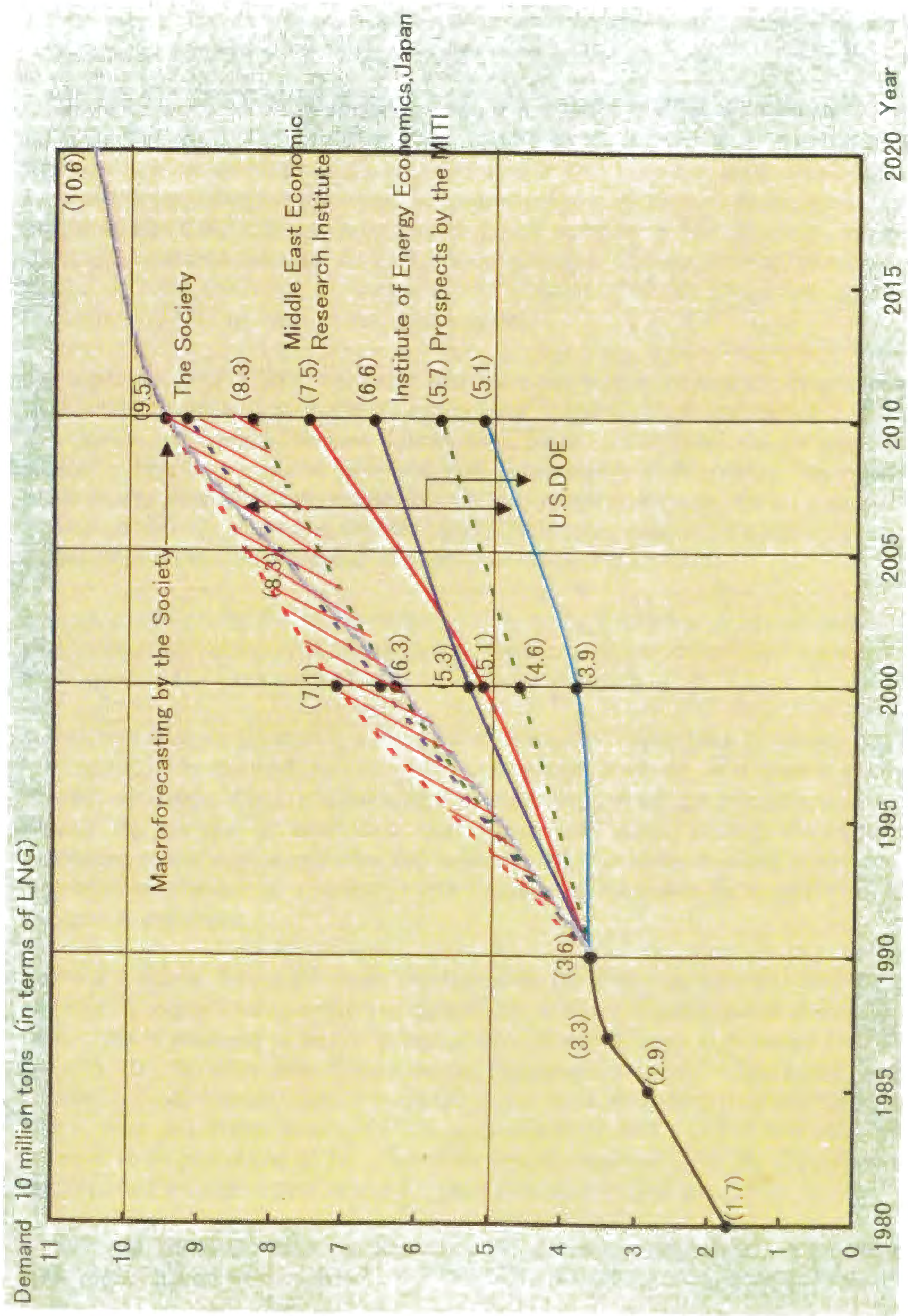


Figure 2.2: Natural gas demand forecasting in Japan

At present, difficult price negotiations are continuing between producing countries and purchasers regarding the price of LNG produced from new gas fields. This situation has arisen due to the impending exhaustion (in about 2004) of the Arun gas field in Indonesia, which has long been the world's largest LNG production base. The influence of this exhaustion is considerable because 4 million tons out of the total import quantity of 8.8 million tons covered by the 1973 contract comes from Arun.

Indonesia began to develop a natural gas field in the East China Sea with estimated reserves of 150 trillion cubic feet (4 trillion cubic metres) as an alternative to the Arun gas field. However, the development and production cost of LNG from this large natural gas field is expected to be enormous because it is understood that 70 percent of the produced gas is carbon dioxide (CO_2). As the processing costs will be added to the LNG price, importers of LNG from Indonesia calculate that the current CIF price of about US$3.20 per million BTU (1 BTU = 0.252 kilocalorie) in Japan will jump to about US$5.00. Therefore, Japan will be forced to buy LNG at a 50 percent higher price.

The importers of LNG from Southeast Asia are expanding to include the Republic of Korea (ROK) and Taiwan, adding to Japan's monopoly in the past. (ROK imported 5.8 million tons and Taiwan imported 2.2 million tons in fiscal 1994.) In addition, natural gas producers such as Indonesia are said to be increasingly aggressive in their pricing. The reason given is that purchasers should appreciate the cleanliness of natural gas and be prepared to pay a premium for such an environmentally acceptable energy source. In addition, the costs of liquefaction plants has increased to over three times that in 1973.

It is not only Indonesia that will increase the LNG price. If all other producers act in concert with Indonesia in raising LNG prices, Japan's energy policies, as well as the nuclear power plant construction problem, will have to radically reviewed.

In view of the above situation it is proposed to change the importation of natural gas in LNG form alone, as in the past, and to import gas through pipelines, as is general in advanced Western countries. It is firmly believed that importing natural gas through pipelines would expand the number of alternative gas sources and would strengthen Japan's price negotiating power against inflexible LNG contracts. Furthermore, it would contribute to the promotion of international cooperation with neighboring Northeast Asian countries, through the natural gas trade.

Figure 2.2 shows that in the study conducted by the "National Pipeline Research Society of Japan", Japan's consumption of natural gas in 2005 is estimated at about 83 million tons. This is assumed to be comprised of 65 - 70 million tons of imported LNG (78 - 84 percent), 10 - 15 million tons of gas imported through pipelines (12 - 18 percent) and about 3 million tons of domestic gas (4 percent). Also, as an extra long-range forecast, Japan's total natural gas consumption in 2020 is estimated at 100 - 110 million tons, which is assumed to be comprised of 70 - 75 million tons of imported LNG, 25 - 30 million tons of gas imported through pipelines and 5 million tons of domestic gas.

What routes will these natural gas pipelines take? On various occasions, the following more likely routes have been proposed. The first is the 1 400 km route from Sakhalin through Hokkaido to Honshu. The Japanese government has given the first priority to this route. The second is the 4 800 km route from East Siberia (Irkutsk and Yakutsk regions) through Mongolia and China to Japan, and the third is the 7 000 km route from Central Asia

(Turkmenistan and Kazakhstan) through the Tarim Basin in China, to Japan. Figure 2.3 shows these proposed routes.

Field trips have been made to some parts of these routes and it has been learned that all of them start from areas where there are huge gas fields drawing considerable international attention at present.

The first project, involving production from an offshore natural gas field located to the north-northeast of Sakhalin Island region is thickly covered with seasonal ice and subjected to severe weather conditions in winter. The second route has the problem that a considerable part of it passes through a permafrost zone creating more than normal difficulties for pipeline design and installation. The third route, which is the longest in total length, passes mostly through a desert zone and must overcome the severe natural conditions.

The project, to deliver natural gas to Japan by pipeline, cannot be commercially materialized unless it is possible to deliver gas to purchasers at lower prices than the imported LNG. Therefore, it will be necessary for the project to receive considerable international cooperation, such as has never been technically or economically experienced by Japan.

Among the various tasks regarding these routes, pipeline laying in permafrost is totally unprecedented in Japan. The Trans-Alaskan oil pipeline constructed by Western technology in 1972 is well known throughout the world. Later on, the Norman Wells oil pipeline in the northern part of Canada is known as the most successful pipeline buried in permafrost. In Russia, on the other hand, there is the natural gas pipeline system from West Siberia passing through permafrost and then continuing 7 000 km to East Europe.

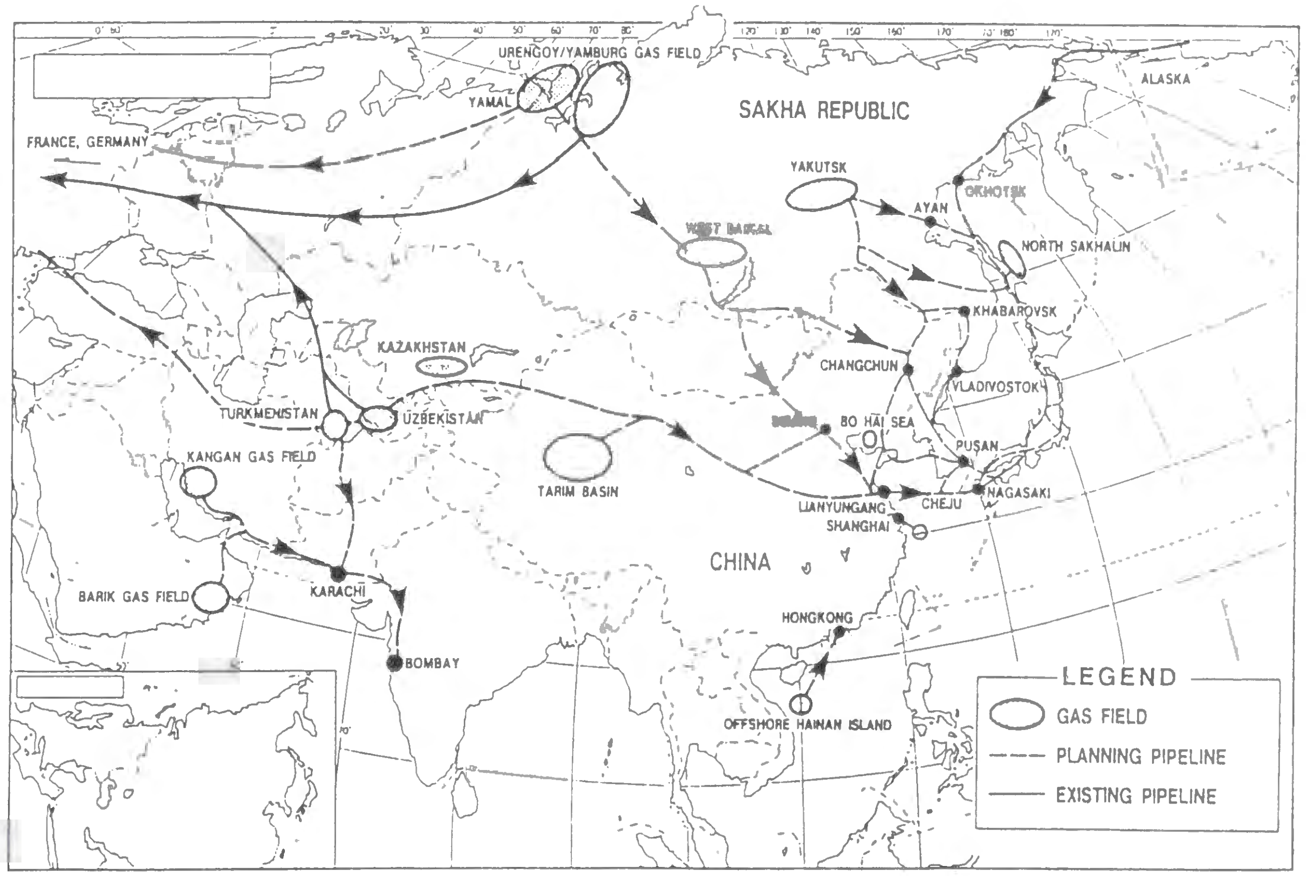


Figure 2.3: Northeast Asian natural gas pipeline network (Hirata, 1996)

3.0 PERMAFROST DISTRIBUTION AND CHARACTERISTICS

3.1 PERMAFROST DISTRIBUTION

3.1.1 Global Permafrost Distribution

Permafrost, or perennially frozen ground, is defined as "soil or rock that remains below 0°C for at least two years". Permafrost regions underlay over 20% of the earth's surface, most of which exists in Russia (65% of land surface) and Canada (50% of surface). Figure 3.1 shows the distribution of permafrost in the Northern Hemisphere (Andersland and Anderson, 1987).

There are four main permafrost regions, defined as follows:

- **continuous**, which is the coldest, most northerly permafrost. It is so defined because permafrost exists beneath all land surfaces in this region. Beneath rivers and lakes, however, there will be unfrozen ground, the depth of which will depend on the size and the mean temperature of the water body;

- **discontinuous,** where permafrost exists along with areas of unfrozen ground. The proportion of frozen ground decreases southwards. In the more southerly portion, the permafrost zones can be quite scattered;

- **sporadic,** where there are only isolated "islands" of permafrost, typically associated with peatlands;

- **alpine,** which is directly related to altitude, and is the prime cause of the southerly extension of permafrost in western North America and in northeast Asia.

The occurrence and distribution of permafrost is primarily related to climatic factors. The mean annual ground temperature is typically 2 to 4°C warmer than the mean annual air temperature, depending on the surface vegetation and drainage. The boundary between the continuous and discontinuous permafrost is close to the -8°C air temperature isotherm. The southern limit of permafrost has been roughly defined by the -1°C isotherm, however, in this southerly region the actual occurrence is highly influenced by the vegetation, surface drainage and aspect (ie., permafrost is more prevalent on north facing slopes).

The maximum observed permafrost thickness in northern Canada is 720 m, however, thicknesses to 1,000 m have been projected by Judge (1973). The maximum thickness in Russia is reported to be 1 500 m. Figure 3.2 shows a north-south section through the North American permafrost. The thickness decreases southwards and may be less than a metre thick in the isolated islands. The thickness of the seasonal thaw zone, or active layer, is generally very thin in the most northerly permafrost and increases in thickness southwards. Exceptions include areas in the high arctic where there is little vegetation, resulting in up to 2 m of seasonal thaw, and areas of dry peat in the southerly sporadic permafrost, where the active layer may be less than 0.5 m.

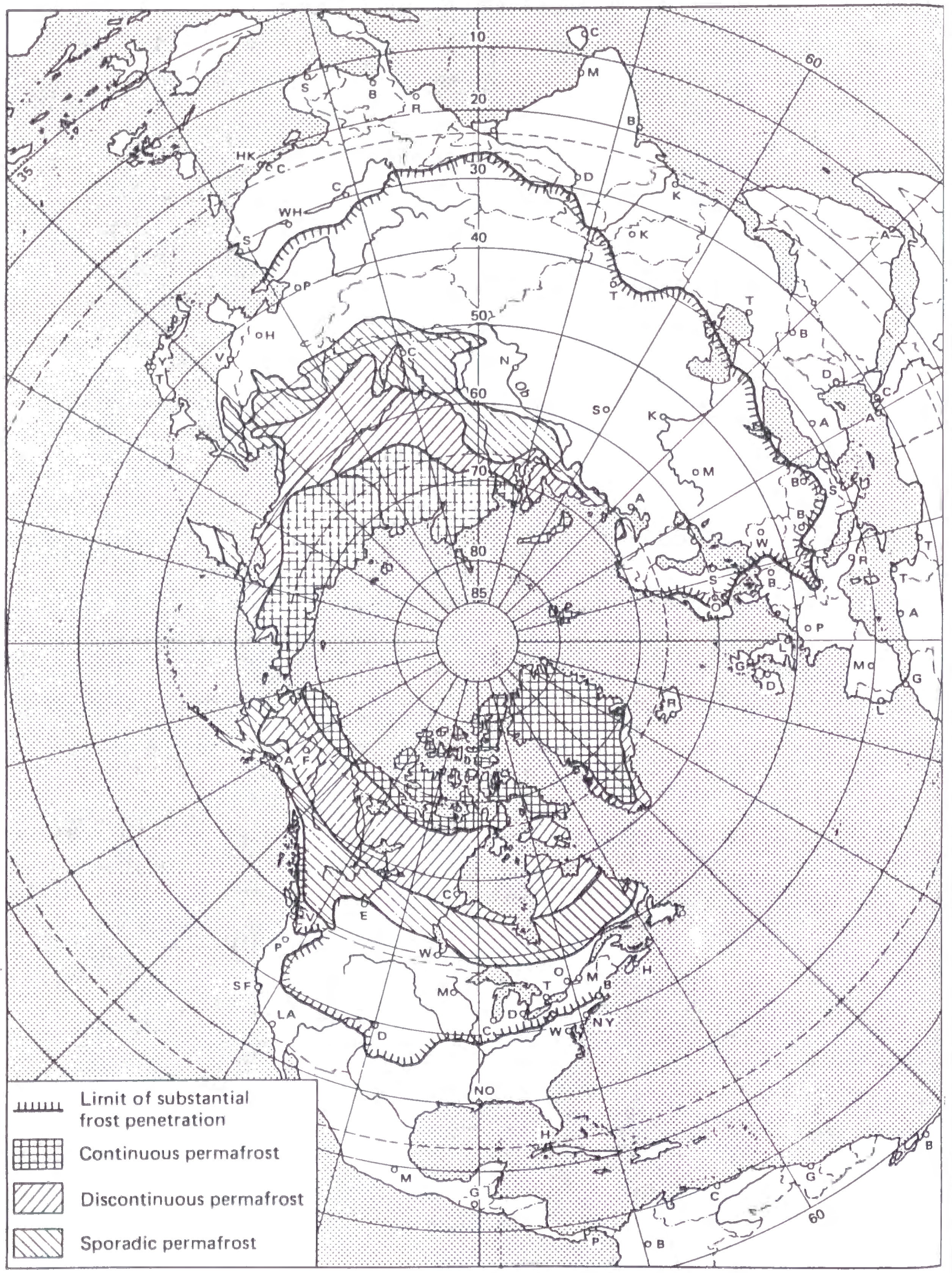


Figure 3.1: Cold regions of the Northern Hemisphere (Andersland and Anderson, 1987)

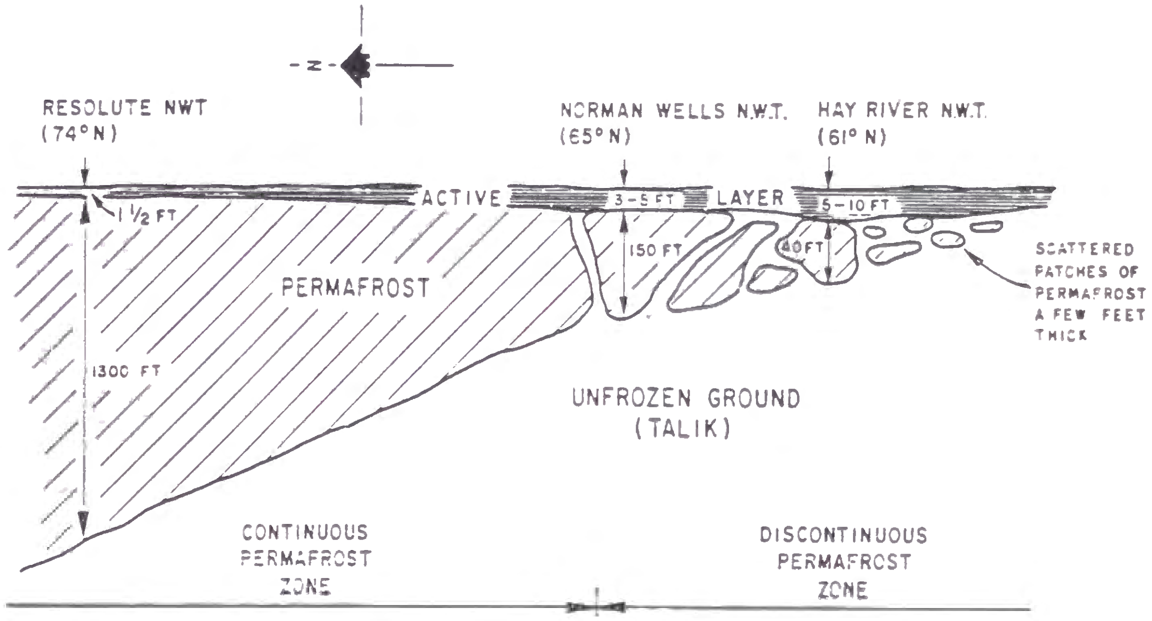


Figure 3.2: Typical vertical distribution and thickness of permafrost (from Johnston, 1981)

3.1.2 Physical Characteristic

3.1.2.1 Terrain Features

There are several distinctive terrain features, exclusive to permafrost regions that indicate particular local or regional terrain conditions. Some of the more significant features that may impact development are identified below, along with their potential engineering significance:

- **thermokarst** - uneven terrain resulting from the irregular thawing of ice-rich ground and subsequent thaw settlement; lakes typically form in the depressions; indicative of ice-rich fine-grained terrain and generally a degrading permafrost sensitive to disturbance.

- **ice wedge polygons** - wedges of ice form in thermal contraction cracks in the ground with the process increasing the ice wedge to as much as 3 m wide at the surface and up to 10 m deep. The thermal contraction of a large area creates a polygonal pattern of wedges between 15 and 30 m across. Polygons can occur in peat or mineral soil. Development must recognize potential for extreme local concentrations of ice. Polygons usually occur in relatively cold permafrost and therefore are not especially sensitive to disturbance during development.

- **peat plateaus** - typically flat-topped areas of relatively dry peat rising above a generally wet peat area. The plateau contains significant segregated ice. In more southern permafrost regions, the permafrost is typically only present beneath the plateaus. Development must anticipate the variable and highly sensitive ground ice conditions.

3.1.2.2 Ice Contents

As a general rule, coarse-grained sands and gravels tend to contain low to moderate excess ice (i.e., in excess of frozen water in the voids between soil particles). Fine-grained silts and clays tend to contain considerable excess ice especially in the upper 3 to 5 m. Ice can also exist in bedrock, most commonly in sedimentary deposits and also in cracks and fissures in any rock.

In finer-grained soils, ice is often in the form of segregational ice lenses, as shown on Figure 3.3. A complete classification system for ground ice is given by Pihlainen and Johnston (1963). Massive ice, in the order of meters thick is common in the Mackenzie delta area. Some may be a result of buried ice. However, others are extreme cases of segregational ice, as in ice-cored ridges (Dallimore and Wolfe, 1988) and pingos (MacKay, 1973).

Figure 3.3: Photo of ice-rich exposure

The impact of ground-ice on development projects can be significant. Even if the ground is preserved in the frozen state, soil with excess ice exhibits long-term deformations (creep) under sufficiently high sustained loads. This limitation in the load bearing capacity of ice-rich permafrost is particularly significant where the mean annual ground temperatures are warmer than -5°C or if the pore-ice is saline.

The consequences of thermal degradation in ice-rich permafrost include thaw settlement and thaw instability on slopes. Again, the warmer the permafrost, the more readily degradation can occur and the more costly the preservation measures.

3.1.2.3 Unfrozen Moisture Content

At temperatures close to the freezing point, the moisture in the soil pores may not all be frozen (i.e., in the form of ice). There may be a significant proportion of the total moisture that is not frozen, depending on the actual temperature and the fineness of the soil. The unfrozen moisture content can be of particular significance in permafrost which is warmer than -2°C. Figure 3.4 shows some typical unfrozen moisture content relationships for certain soil types. The finest soils have the greatest proportion of unfrozen moisture. The prime reason for this phenomena is the surface forces, which are mainly capillary action and adsorption to the soil particles, as well as some physico-chemical factors. In effect, the moisture that is in immediate contact with the soil particles does not freeze until colder temperatures are reached, and in the finer soils, a definite proportion of the moisture may remain unfrozen, even at the colder natural permafrost temperatures experienced in the high arctic (-10 to -15°C).

This unfrozen moisture content has a considerable influence on the physical and thermal properties of permafrost. It can explain, for example, how the strength of frozen soil increases significantly between initial freezing and at least -10°C. There is also a major influence on the deformation response to loading at the relatively warm frozen temperatures.

It should be noted that the Russian classification for frozen soils recognizes the influence of the unfrozen moisture content, as follows:

<u>hard frozen</u> - soils that are firmly cemented with ice, are subject to a relatively brittle failure and exhibit practically no compression under loads exerted by the structure; hard frozen soil include sandy and clayey soils, if their temperatures are below the following values:

- silty sands	-0.3°C
- sandy loams	-0.6°C
- clay loams	-1.0°C
- clays	-1.5°C

<u>plastic frozen</u> - soils cemented by ice but with viscous properties (due to a high unfrozen water content) such that they are subject to compression under loads exerted by the structure; plastic frozen soils include sandy and clay soils:

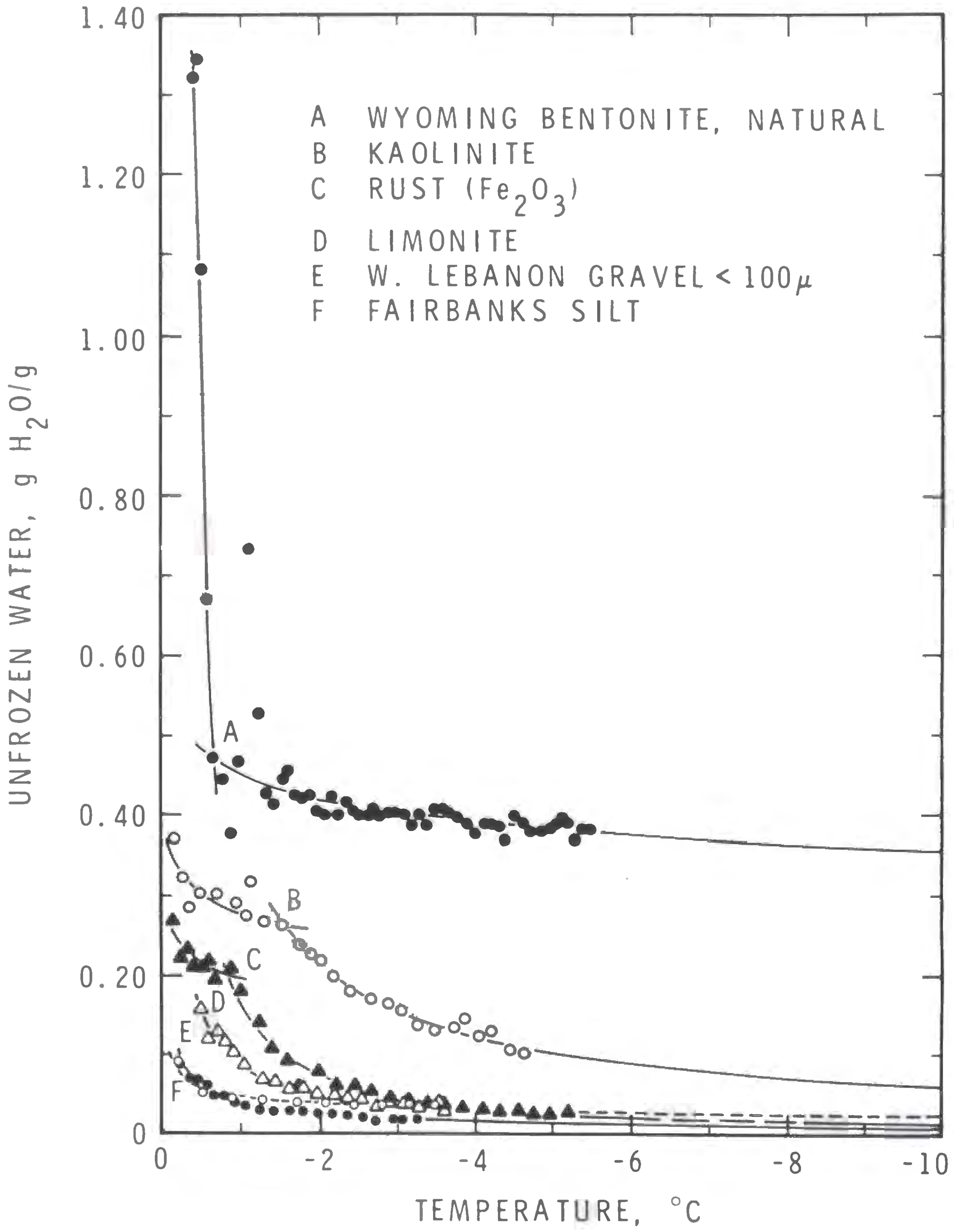


Figure 3.4: Variation of unfrozen water content with temperature for six representative soils and soil constituents (After Anderson and Morgenstern, 1973)

<u>friable frozen</u> - sandy and coarse-grained soil not cemented by ice due to a low moisture content.

3.1.3 Northeast Asian Permafrost Distribution

Most of the East Siberian gas fields are situated in the Krasnoyarsk-Irkutsk-Sakha region, which are located mostly in the discontinuous permafrost area, shown on Figure 3.5. Any prospective routes for oil and gas pipelines to the Far East (Russia, Korea or China) will have to cross territories with discontinuous, sporadic and alpine permafrost in Russia, Mongolia or northeast China. The following sections describe the geocryological conditions of Southeast Russia, Mongolia and Northeast China in the vicinity of prospective pipeline routes to the Far East.

3.1.3.1 <u>Southeast Russia</u>

There are 9 geocryological regions in the southeast portion of the Russian territory, from the Irkutsk region to the Far East, as shown on Figure 3.6.

- **Angaro - Lenskiy geocryological region.** This region occupies the South - Eastern part of the Srednesibirskoye ploskogorye. The mountain elevations rise to between 800 and 1100 m, and the river valleys drop down to 300 to 400 m. Frozen soils are encountered, as a rule, in swampy valleys of the rivers, shaded ravines, at the bases of north facing wooded slopes. The percentage of permafrost increases in direction from South - West (5 - 25%) to North - East (20 -50%). The Northeastern part of the region has permafrost at the flat ridges.

  The minimum mean annual temperature of permafrost (-1 to -2 °C) is typical for the swampy river valleys. The frozen mountain ridges have the mean annual temperature of -0.5 to -0.7 °C; the rest permafrost has a mean annual temperature around -0 to -0.3 °C.

- **Tunguskiy geocryological region.** This region is located to the North of the Angaro - Lenskiy geocryological region. The elevations vary from 250 to 600 m. The river valleys are generally wide with shallow slopes. Frozen soils can be encountered in any kind of relief, in swampy and mossy sites, shaded slopes (shallow and steep). The permafrost area reaches 20 -35%.

  The mean annual temperature of the permafrost increases in direction from the valley bottoms to mountain ridges within a range of -1.0 to -0.2 °C.

- **Baikalo - Patomskiy geocryological region.** This region has mountain relief with elevations of 900 - 1900 m. Frozen soils can be encountered in any relief: slopes, ridges, river valleys. The taliks are found in the tectonic active zones and in the some large river valleys. The talik zones have area approximately 0.5 - 1.0 km². The mean annual permafrost temperature varies from -1 to -3 °C.

Figure 3.5: Distribution Of Permafrost In Northern Asia

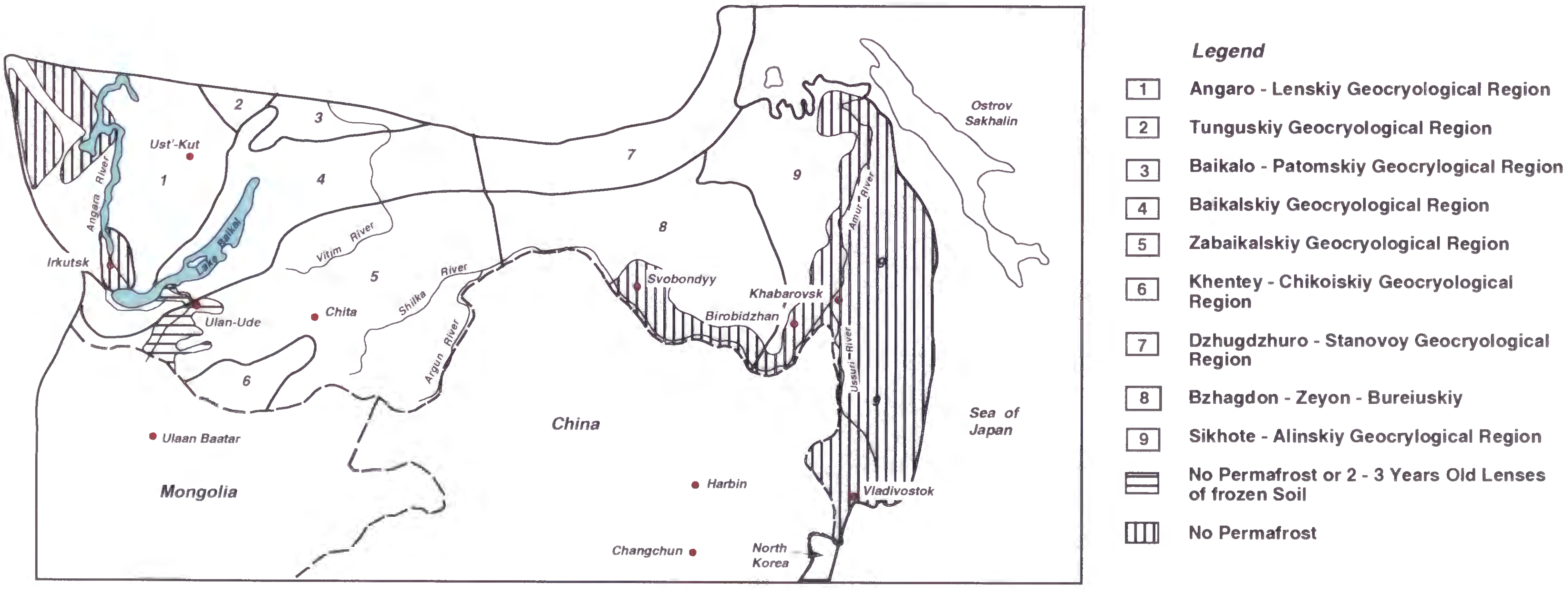


Figure 3.6: Geocryological Regions of Russian Southern Siberia and Far East

- **Baikalskiy geocryological region**. This region occupies the territory of the Baikalo - Stanovoe Nagorye with the elevations from 500 to 3000 m. These are the mountains which surround Lake Baikal and extend in a northeastern direction. This description comprises only part of the region adjacent to Lake Baikal. This territory has a relatively soft climate and discontinuous or sporadic distribution of permafrost. The southern coast of Lake Baikal has no permafrost. The slopes, facing the lake have isolated "islands" of permafrost, with an area of up to 1 to 2 km^2. In the mountains, the permafrost occurs on north facing slopes; the total area of frozen soils reaches 20%. The permafrost area in depressions and the river valleys is slightly less and, as a rule, consists 10 -15%. Permafrost may also be encountered near the base of slopes and at river terraces. The mean annual permafrost temperature equals -0.2 to -0.5 °C in the mountains; in the depressions and the river valleys, the temperatures are slightly warmer (0 to -0.3 °C).

- **Zabaikalskiy geocryological region**. It is the mountain region which extends from the Selenga river to Skovorodino in a southwest - northeast direction. The main feature of this region is the lower elevations in comparison with the adjacent regions: Baikalskiy (to the north) and Khentey - Chikoiskiy (to the south). The most part of the region has the elevations 800 to 1200 m. The bottoms of depressions and the river valleys have the elevations 500 - 700 m.

  The region is distinguished by variable geocryological conditions which depend on relief, face of a slope, geological structure and hydrogeological conditions. The permafrost occurs on north facing slopes, the swampy and mossy areas. The permafrost occupies not more than 10% of a total territory.

  A small thickness of snow cover is the reason the permafrost exists with relatively high a mean annual air temperature (-0.5 to -3.0 °C). A natural phenomenon exists, where the mean annual permafrost temperature increases with elevation. This circumstance is created by a natural winter air temperature inversion, such that the cold winter is located in lowest areas of relief. The minimum mean annual permafrost temperature equals -1 to -1.5 °C occurs on steep north facing slopes and the narrow swampy bottoms of the river valleys. The shallow, north facing slopes have temperatures within the range of -0.2 to -0.5 °C.

- **Khentey - Chikoiskiy geocryological region**. This region is located in the southeastern direction from Ulan - Ude, near the Mongolian border. It is a massive upland with elevations up to 1200 - 1600 m. The percentage of permafrost increases with elevation: the interval 1200 - 1600 m has 50 -80% permafrost; the percent of permafrost within interval 800 - 1200 m varies from 30 to 50%; the elevations 800 m and lower have 10 - 30% of permafrost. The latter geocryological zone has permafrost at the bases of north facing slopes and the shaded bottoms of small valleys. The mean annual permafrost temperature equals -0.1 to -0.5 °C. The most part of the Khentey - Chikoiskiy geocryological region has elevations 1200 - 1600 m. Permafrost can be encountered at any kind of relief with the exception of flood plains of big rivers and south facing slopes. The mean annual permafrost temperature varies from -1 to -3 °C.

- **Dzhugdzhuro - Stanovoy geocryological region**. This region comprises the Dzhugdzhur ridge and southern part of the Stanovoy ridge and Zeya depression. The ridges have elevations 700 - 2400 m. The elevations increase in the direction from west to east. The permafrost is almost continuous for mountain areas with elevations higher than 1100 - 1200 m. Taliks are encountered only at flat ridges and south facing slopes.

Taliks in mountains with elevations 700 - 1100 m can be encountered at any relief. As a rule, the taliks are found in coarse soils which have a high permeability. The swampy and mossy bottoms of valleys , generally, are frozen irrespectively of elevations. The Zeya depression has no permafrost.

The range of mean annual permafrost temperatures is -1 to -6 °C. The lowest temperatures (-2 to -6 ° C) are typical for the mountain slopes and ridges with elevations higher than 1100 - 1200 m. For the elevations from 700 to 1100 m, the permafrost has mean annual temperatures of -1 to -2 °C. The swampy valleys have relatively cold permafrost (-2 to -3 °C).

- **Dzhagda - Zeya - Bureinskiy geocryological region**. The narrow Dzhagda ridge occupies the northern part of the region. The base of the ridge has elevations 600 - 800 m and the peaks reach elevations up to 1600 m. The remainder of the territory consists of alternating medium high mountains with elevations 600 - 1200 m, and hilly plains with the elevations 200 - 600 m.

The predominant part of the Dzhagda ridge with elevations below 900 m has no permafrost. The sporadic frozen soils can be encountered at the base of north facing slopes. No data is available on the permafrost distribution in the mountains which have elevations higher than 900 m, however, predictions show that the mean annual soil temperatures could be -1 to -2 °C at elevations of 900 - 1300 m, and -2 to -5 °C at elevations of 1300 m and higher. The distribution of permafrost in the mountains depends on the moisture content and grain size of the surficial deposits. The plains have isolated "islands" of permafrost at the swampy - mossy and shaded areas of the slopes and valleys.

The mean annual temperature of the sporadic permafrost in Dzhagda ridge varies from -0.5 to -1.0 °C at the slopes; the very rare islands of permafrost at bottoms of the river valleys temperatures close to 0 °C. The mountain areas of the rest territory have the mean annual permafrost temperature -1 to -2 °C; the shaded swampy and mossy areas of valleys and slopes at the plain have the permafrost temperature up to -1 °C.

- **Sikhote - Alinskiy geocryological region**. The description of geocryological conditions is provided only for the southern part of the region, which occupies the Sikhote - Alin ridge. The ridge reaches elevations of 800 to 1200 m and several peaks with elevations 1800 - 2090 m. Depressions have elevations not higher than 100 m. There is a significant network of rivers and many lakes in the depressions. Permafrost is encountered only in the mountains with elevations higher than 1000 m. At elevations from 1000 to 1300 m, the permafrost occurs on north facing slopes. At elevations higher than 1300 m, the permafrost can be found more extensively, including south facing slopes.

The mean annual permafrost temperature varies from zero to -2 °C; the peaks with elevations around 2000 m have temperatures of approximately -3 °C.

3.1.3.2 Mongolia

Permafrost in Mongolia occupies the Mongolian Altay, Hangay, Hentey mountains and the Orkhon - Selenginskoy middle land, as shown on Figure 3.7. Prospective routes of the gas pipeline cross only the Hentey mountains and Orkhon - Selemginskoe middle land, and details are provided for these regions. The geocryological zone with essentially continuous permafrost occurs in the Hentey mountains at elevations higher than 1650 to 1750 m in the northern part of the zone, and at elevations higher than 2000 m in the southern part of the zone.

The typical mean annual permafrost temperature varies from -1 to -4 °C. The wide range of the permafrost temperatures is because this geocryological zone has a wide range of elevations (800 to 1000 m). The warmest permafrost temperatures are encountered in this zone at the bottoms of the river valleys in coarse sands.

Discontinuous permafrost occurs at elevations higher than 1400 m at the western and at the northern parts of the Hentey mountains and the Orkhon - Selenginskoe middle land. The mean annual permafrost temperature varies from zero to -1 °C.

The geocryological zone with large islands of permafrost occur between elevations of 1200 and 1400 m at the northern part of the Hentey mountains and the Orkhon - Selenginskoe middle land. The mean annual permafrost temperature is around -1 °C.

A zone of islands of permafrost occurs at elevations from 650 to 670 m at the north of Mongolia and from 900 m at the south. The mean annual permafrost temperature varies from zero to -1 °C.

Frozen soils in the form of lenses of permafrost can be encountered, as a rule, near springs. The area of these lenses is typically less than 500 m^2. The mean annual permafrost temperature is close to 0 °C.

3.1.3.3 China

The permafrost region in Northeast China occupies mainly the Da Hinggan Mountains and the Xiao Hinggan Mountains, north of Harbin, as shown on Figure 3.8. There are three high-altitude permafrost zones: predominantly continuous permafrost; permafrost with isolated taliks; isolated permafrost.

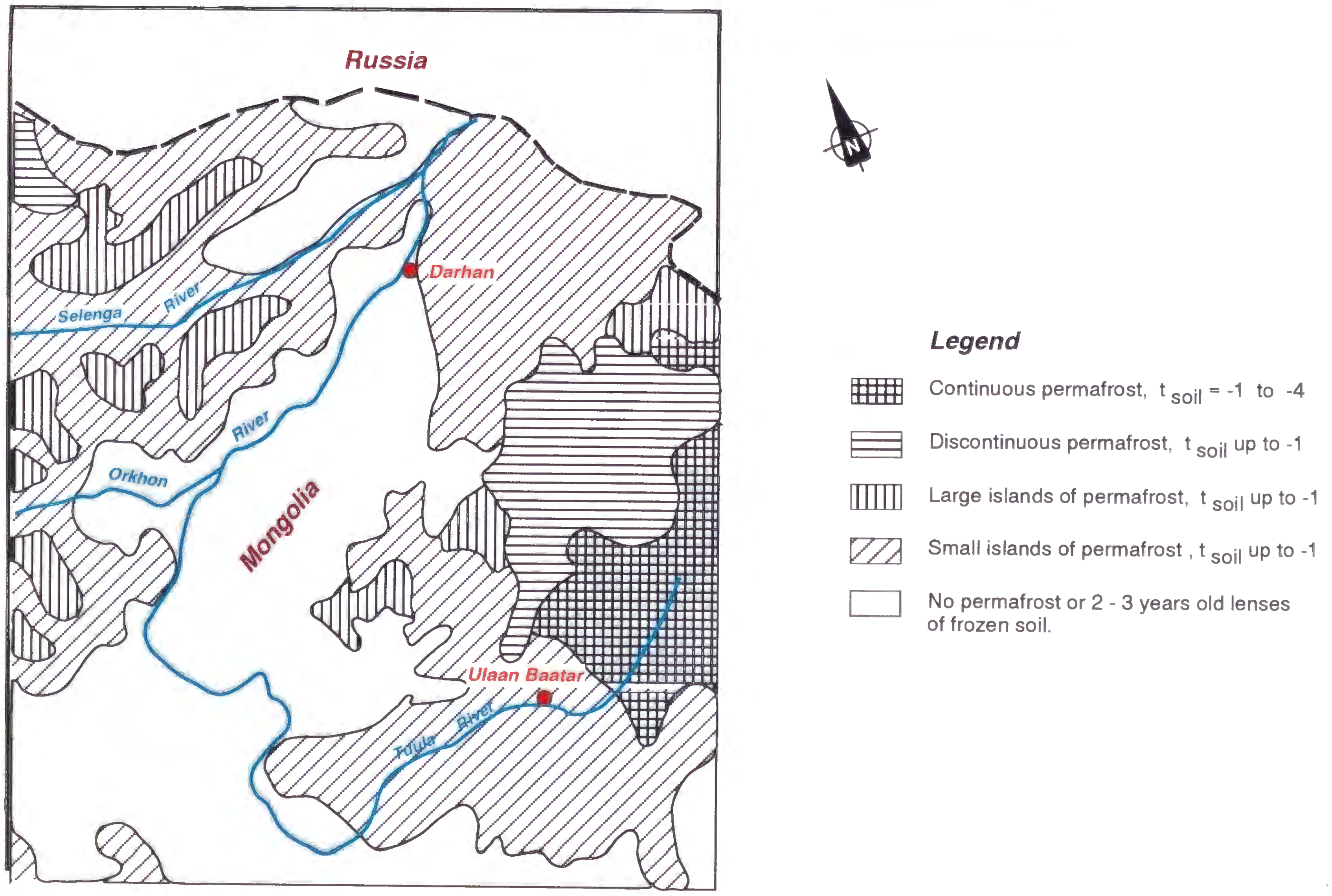


Figure 3.7: Geocryological Map of Mongolia

Scale 1: 32 000 000

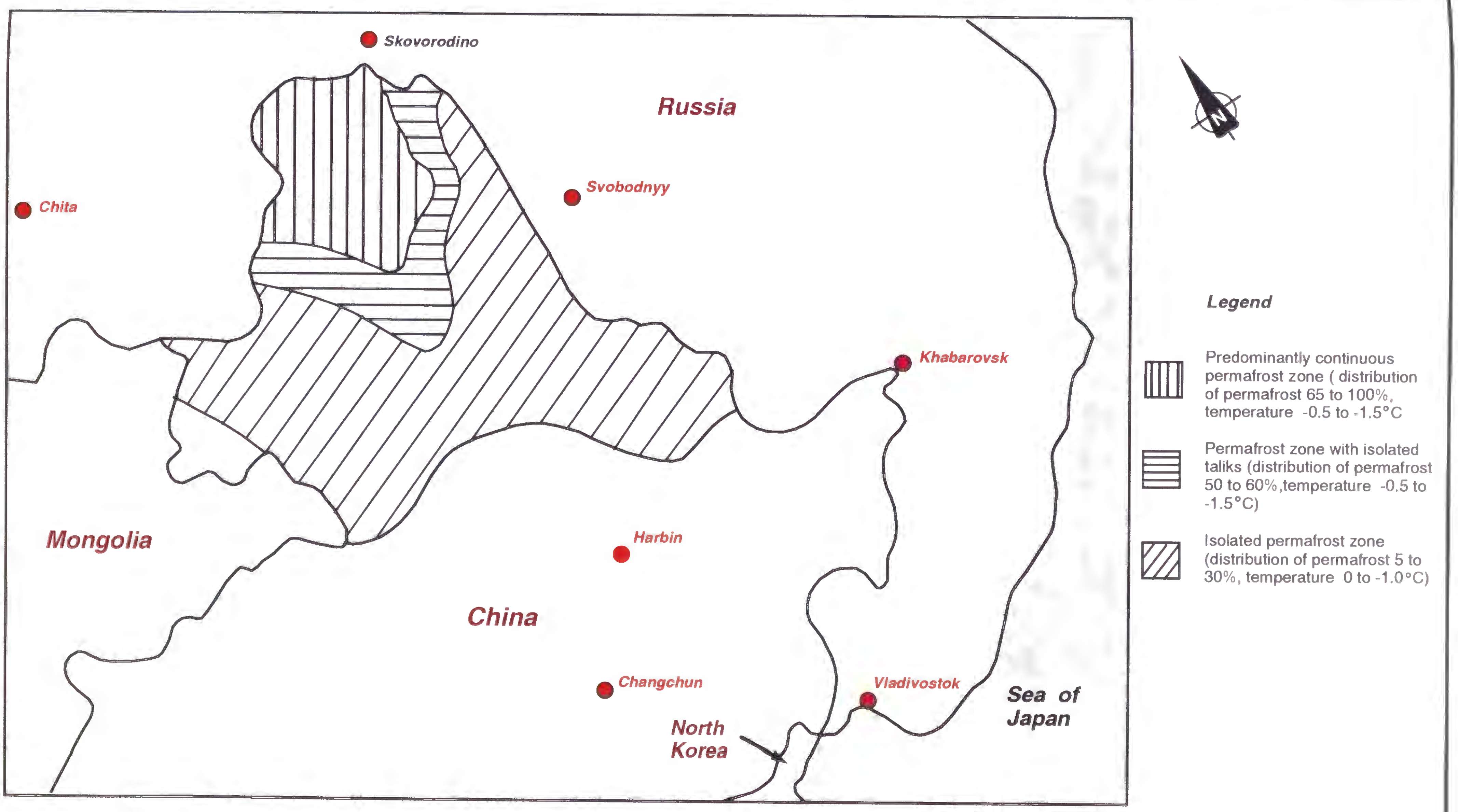


Figure 3.8: Geocrylogical Zones of Northeast China

The Northern part of the region belongs to the predominantly continuous permafrost zone. Distribution of the permafrost is almost 100% in the following terrains: the mossy and swampy bottoms of valleys with clayey soils; alluvial terraces; north facing slopes. These terrains can have high ice content (more than 0.4) and low mean annual permafrost temperatures (-2.0 to -4.0 °C). The percentage of permafrost is down to 65 to 75% on south facing slopes, the terrains with scarce vegetation and the terrains with coarse surficial soils. The ice content is much less in these areas and the mean annual permafrost temperature is -1.0 to -2.0 °C.

The permafrost zone with isolated taliks comprises approximately 60% frozen soils. The mean annual permafrost temperature is -0.5 to -1.5°C. The frozen soils occupy the same terrains as in the predominately continuous permafrost zone.

The isolated permafrost zone has 5 - 30% of frozen soils. The percentage of permafrost varies from place to place. For example, the permafrost islands in the Hulun Buir Plateau occur only in swamps. The percentage of permafrost does not exceed there 10%. The Da Hinggan Mountains have permafrost only on the river terraces and river flood plains and the permafrost reaches to 30%. The percentage of permafrost in the Xiao Hinggan Mountains consists of 20 to 25% and frozen soils occur in the same terrains as in the Da Hinggan Mountains. The mean annual permafrost temperature are around 0 to -1 °C.

3.2 ENGINEERING CHARACTERISTICS, FROZEN AND THAWING SOILS

When considering the most southerly occurrences of discontinuous and sporadic permafrost, it is important to understand that the definition of permafrost is based on temperature alone (ie., ground that remains at or below 0° C for at least two years). Figure 3.9 illustrates this definition based on temperature, as opposed to phase change between water and ice. By this definition, it is important to realize that in cases of saline porewater, for example, permafrost may not be frozen solid, even at -1 or -2 °C. In engineering applications, one must realize that frozen strength does not develop until phase change (to ice) has occurred. Even after phase change, Figure 3.10 demonstrates that there is a considerable further increase in strength of frozen soil (and ice) as the temperature decreases below the freezing temperature.

Given the natural strength of frozen soils, the prime approach to the design of facilities in permafrost regions is to **attempt to keep the ground frozen.** For many types of development this can be achieved by eliminating or reducing any heat input to the ground by means of insulation or intercepting the heat by some active means. Even if the permafrost is prevented from actual thawing, however, any warming too close to the melting point can result in a much weaker soils. A particular phenomenon with permafrost containing significant ice, is that of "creep" deformation under load. Figure 3.11 illustrates creep response to loading. It is normally acceptable to design for applied loads that allow constant, "secondary" creep, however, it is necessary to avoid "tertiary" creep, which is considered as failure. It must be noted that creep occurs under sustained, long term applied loads. The short term strength, under impact loads for example, is considerably greater.

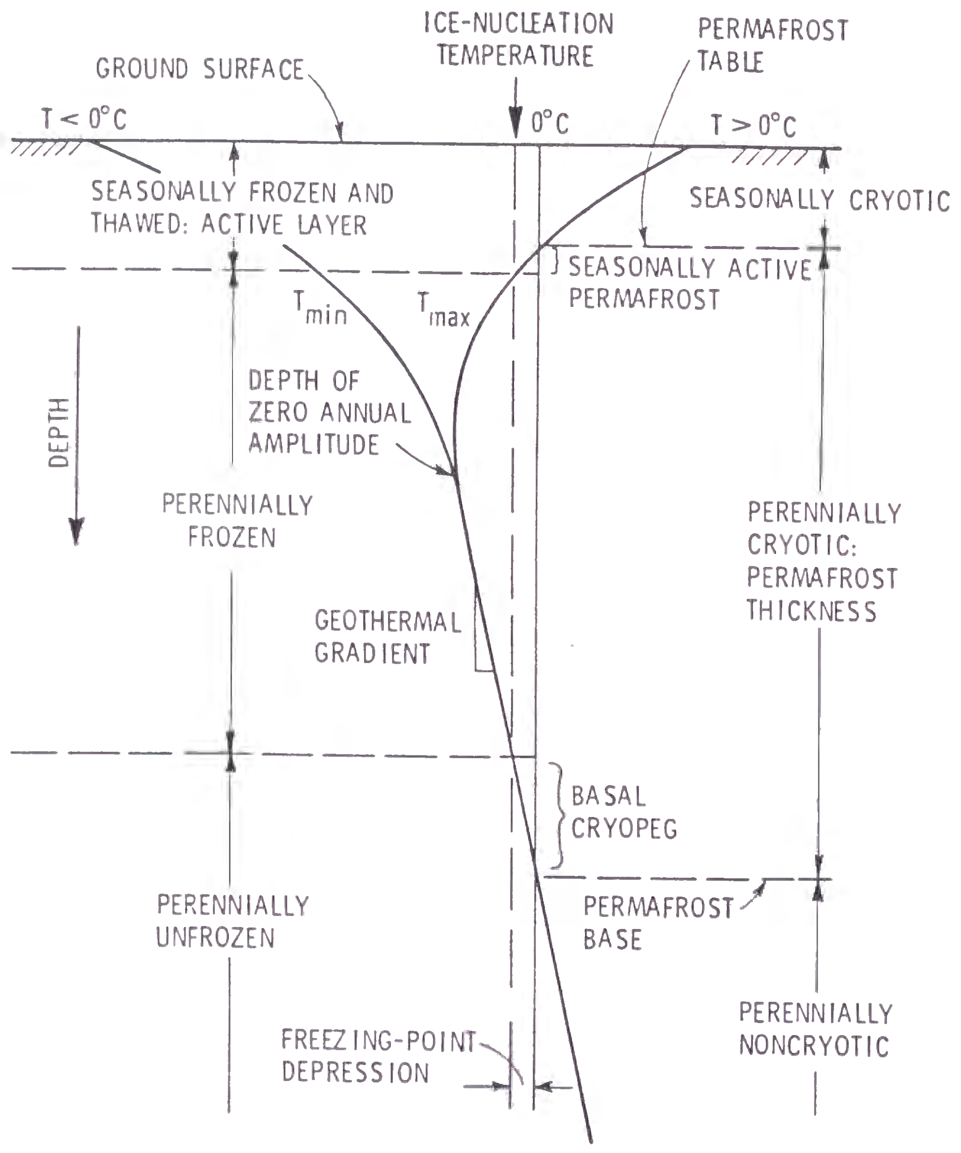


Figure 3.9: Terms used to describe the ground temperature relative to 0°C, and the state of the water versus depth, in a permafrost environment (from NRC, 1988)

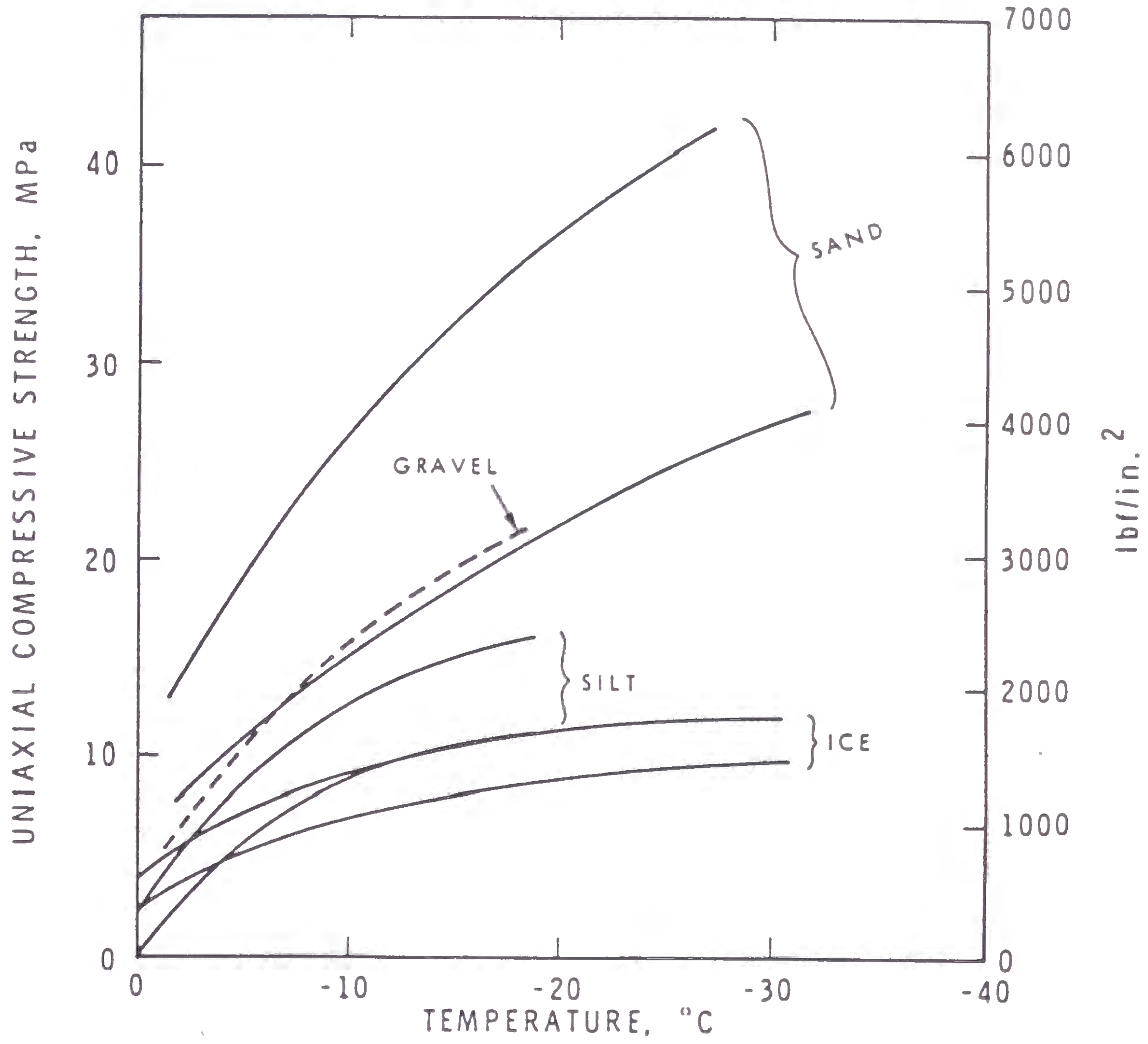


Figure 3.10: Bearing capacity for ice-poor permafrost (from Johnston, 1981; after Mellor, 1972)

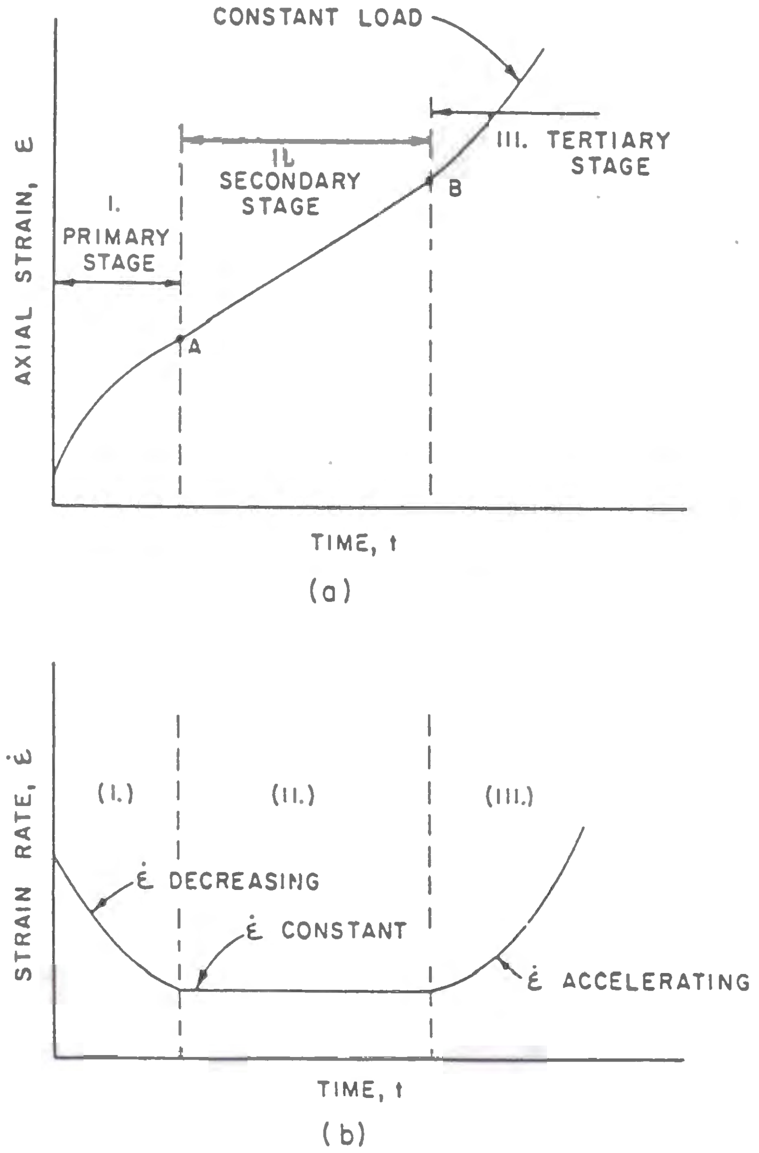


Figure 3.11: Basic creep behaviour (from Phukan, 1985)

For development that will result in thawing of the ground, there are two main issues - thaw settlement and loss of soil strength on thawing. Thaw settlement occurs as the ice in the pores of the frozen soil thaws and the water drains away. The amount of settlement is of course proportional to the amount of excess ice in the soil pores. A relatively simple test procedure has been established in which a cylinder of known frozen volume and bulk density is allowed to thaw under a nominal pressure (e.g., 5 kPa). Two or more subsequent pressure increments (e.g.., 40 and 80 kPa) are applied to the thawed soil. A typical set of results is illustrated on Figure 3.12. This plot enables the interpretation of the settlement due to thawing of the excess alone and the settlement attributable to increases in applied pressure.

By conducting a large number of tests on similar soils, it is possible to establish a correlation between thaw settlement and initial frozen density, as illustrated on Figure 3.13. It is more common (and less expensive) to test for moisture contents of the soil, therefore, thaw settlement correlations can also be established for moisture content. By conducting sufficient tests on the various soil types, it is possible to estimate the thaw settlement potential for all types of permafrost terrain. A series of these thaw settlement correlations for the main soil types is presented on Figures 3.14 and 3.15(Hanna et al. 1983). Other data for coarser soils is presented on Figure 3.16 (Nelson et al, 1983).

There can be a considerable loss of strength on thawing of permafrost. The degree of strength loss is a function of the initial ice content and the fineness of the soil. Nixon and Hanna (1979) have shown (Figure 3.17) that for soils with a frozen bulk density of less than about 1700 kg/m^3, the thawed strength is negligible. The main issue in terms of strength is that as the ice thaws, the excess water must be able to drain away. In coarse sand and gravel, the drainage can occur quite effectively, however, in finer soils the drainage can be seriously impeded. The result is that if water can not readily drain, it will create an increase in the "pore pressure" within the soil matrix. In terms of soil strength and stability, this increase in pore pressure reduces the "effective stress" within the soil mass, and the frictional soil strength is reduced in a proportional manner.

A theoretical representation of this pore pressure has been developed by Morgenstern and Nixon (1971). The thaw consolidation ratio, R, is a dimensionless variable, defined as follows:

$$R = \frac{\alpha}{2\sqrt{(c_v)}}$$

where:

α = rate of advance of the thawing, in terms of the square root of time (cm/sec^{½})

c_v = the conventional coefficient of consolidation of the soil (cm^2/sec), which is closely related to the permeability of the soil.

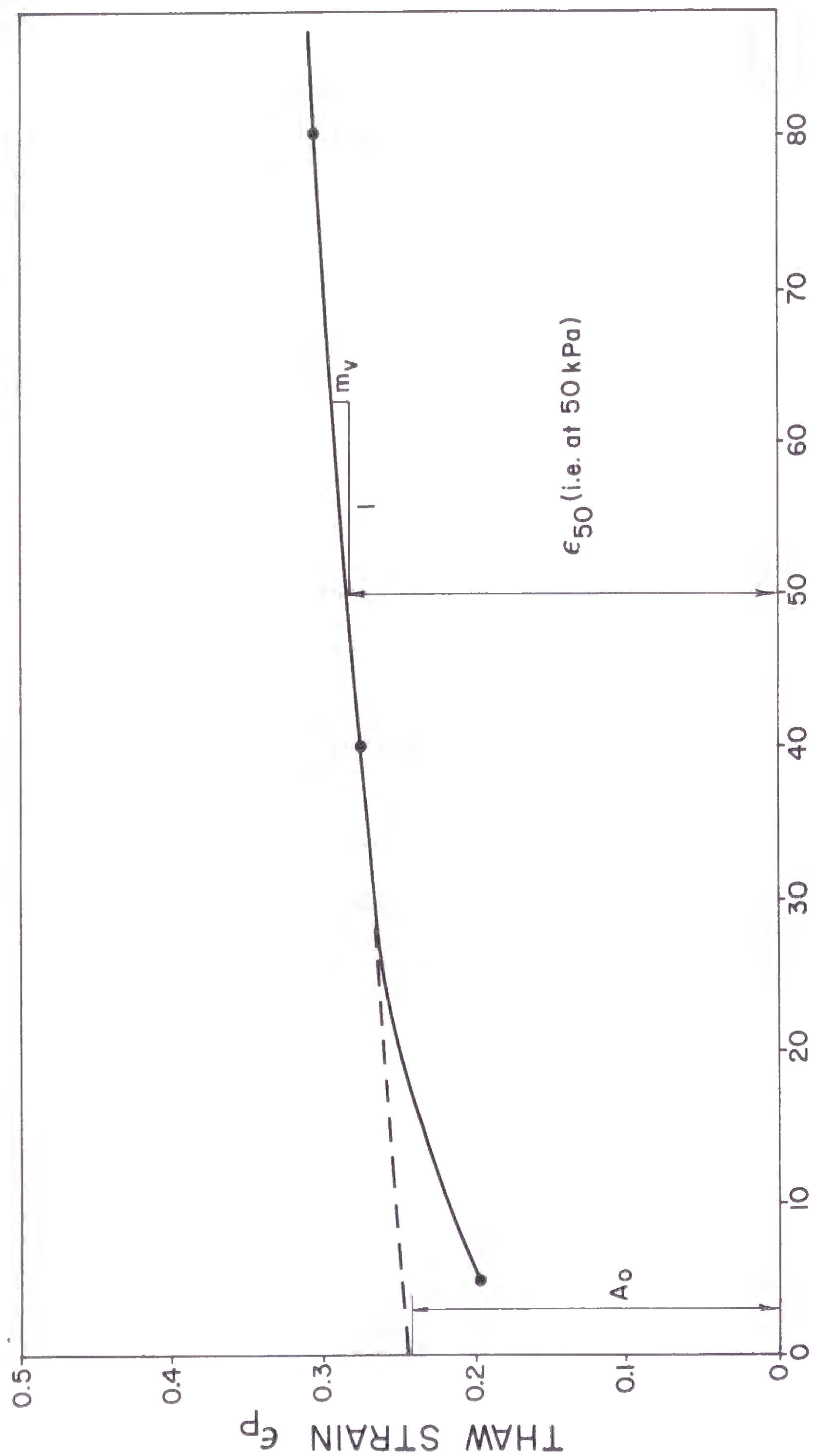


Figure 3.12: Typical thaw settlement test result (from Hanna et al., 1983)

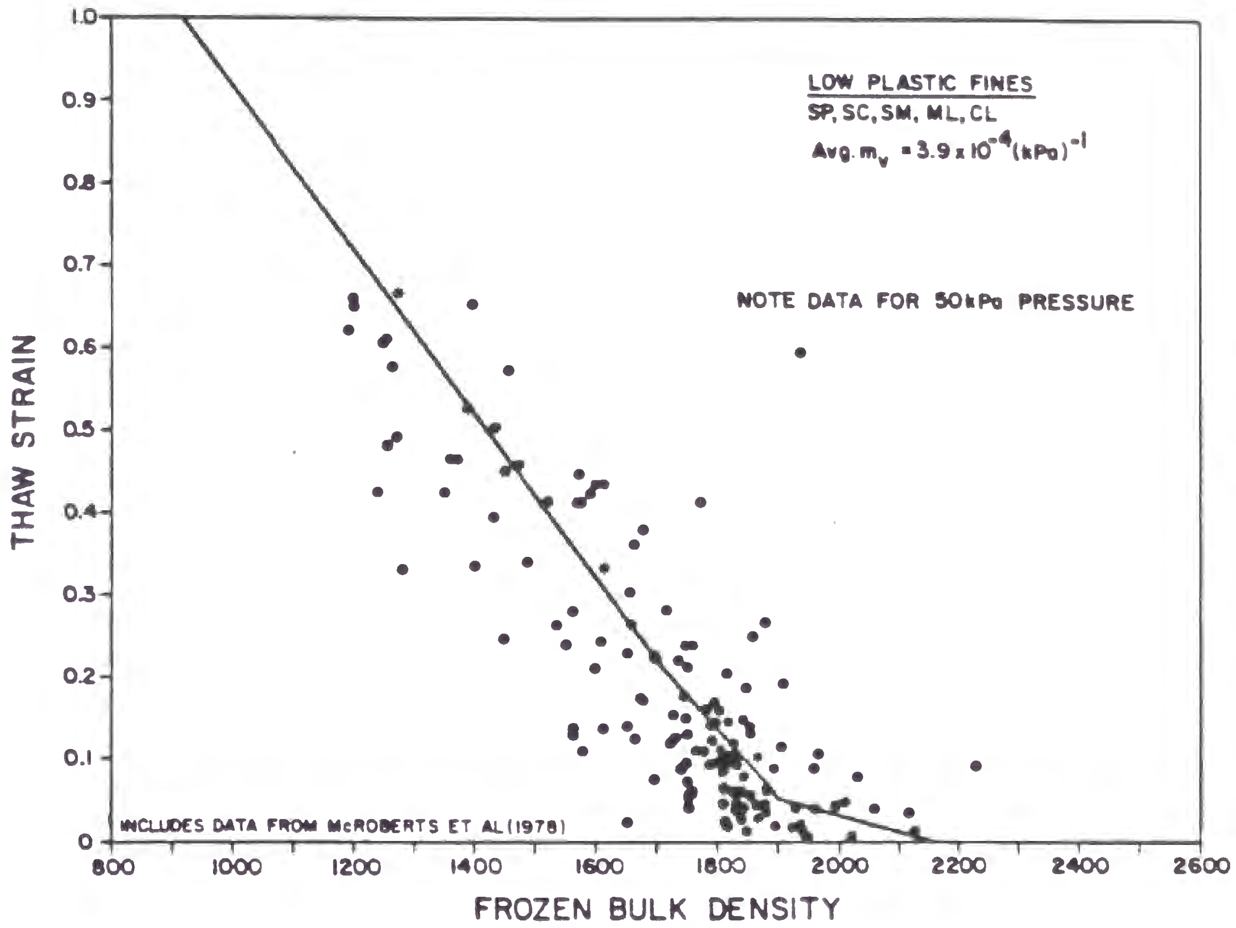


Figure 3.13: Thaw strain-density correlation for low plastic fine-grained soils (from Hanna et al., 1983)

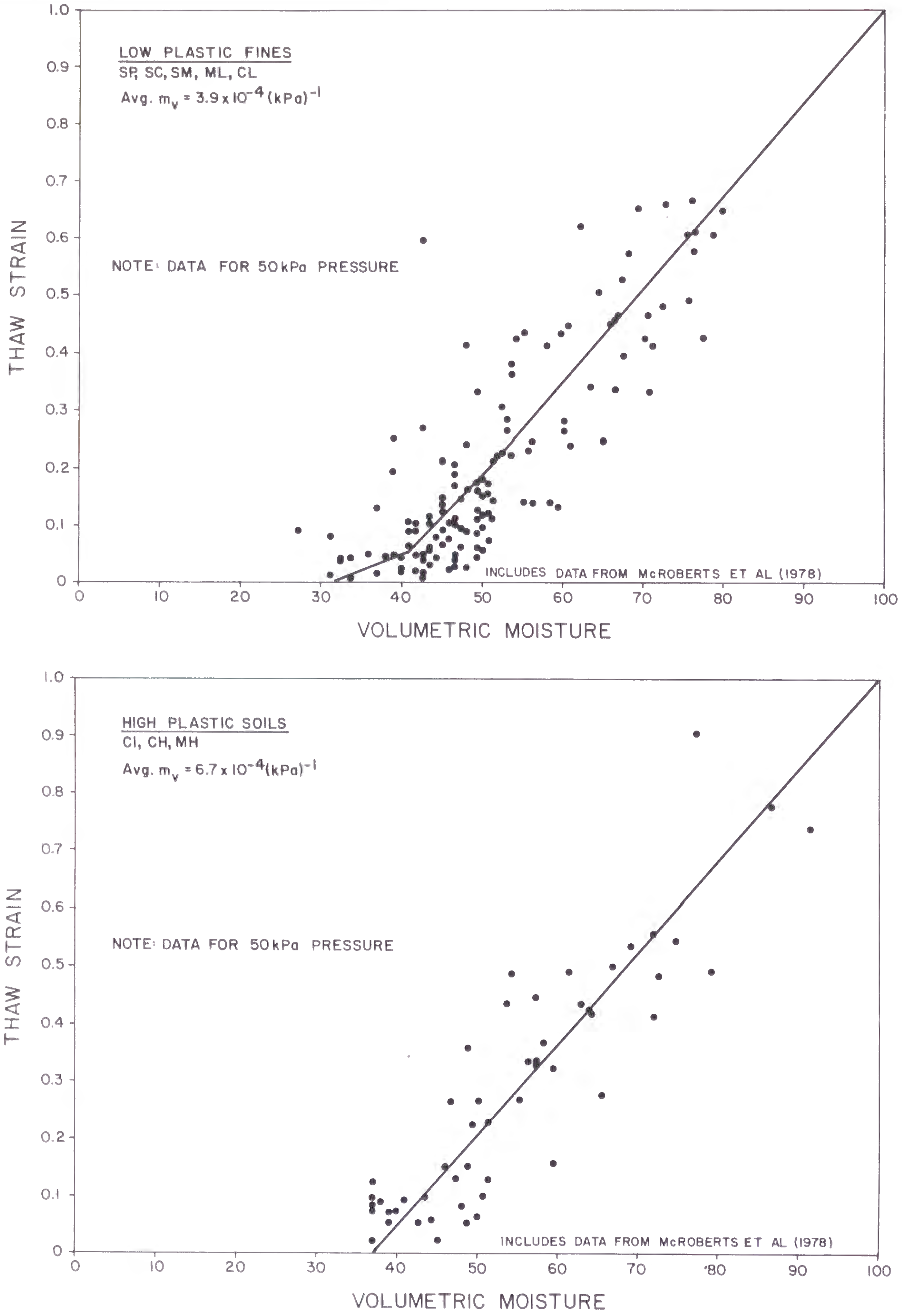


Figure 3.14: Thaw settlement correlation - low and high plastic soils (after Hanna et al., 1983)

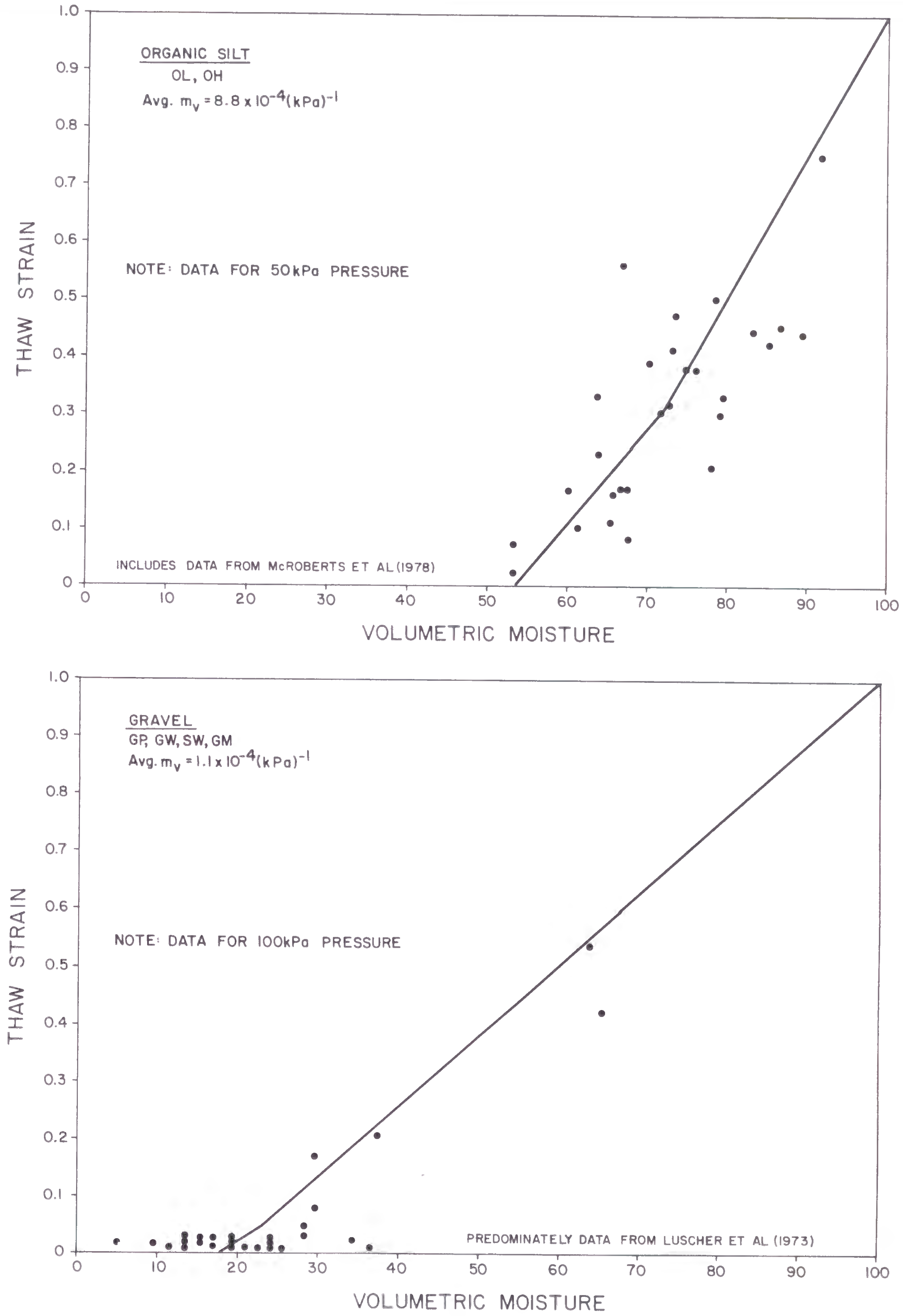


Figure 3.15: Thaw settlement correlation - organic silt and gravel (after Hanna et al., 1983)

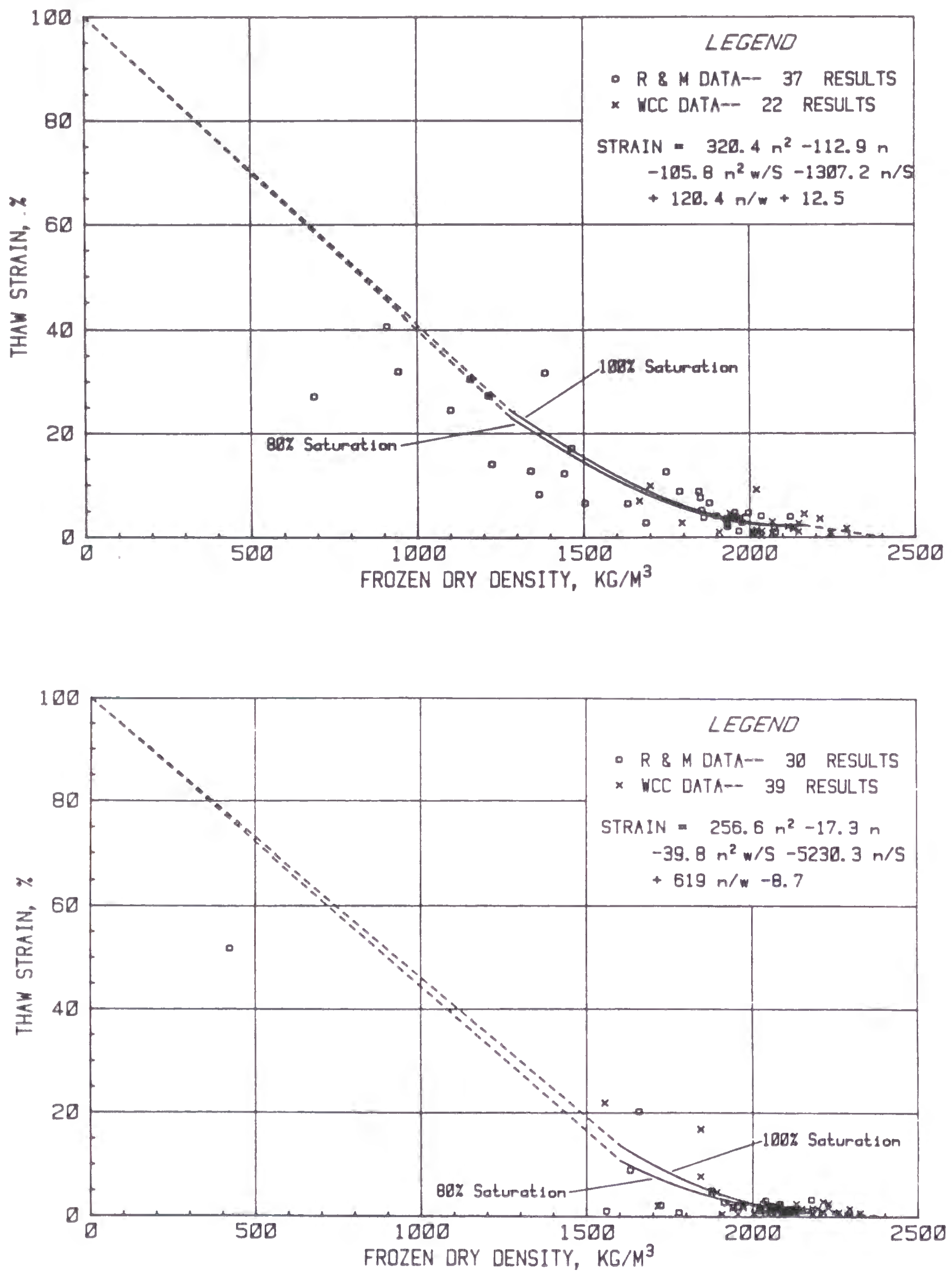


Figure 3.16: Thaw settlement correlations - Alaska Gravels

(From Nelson et al., 1983)

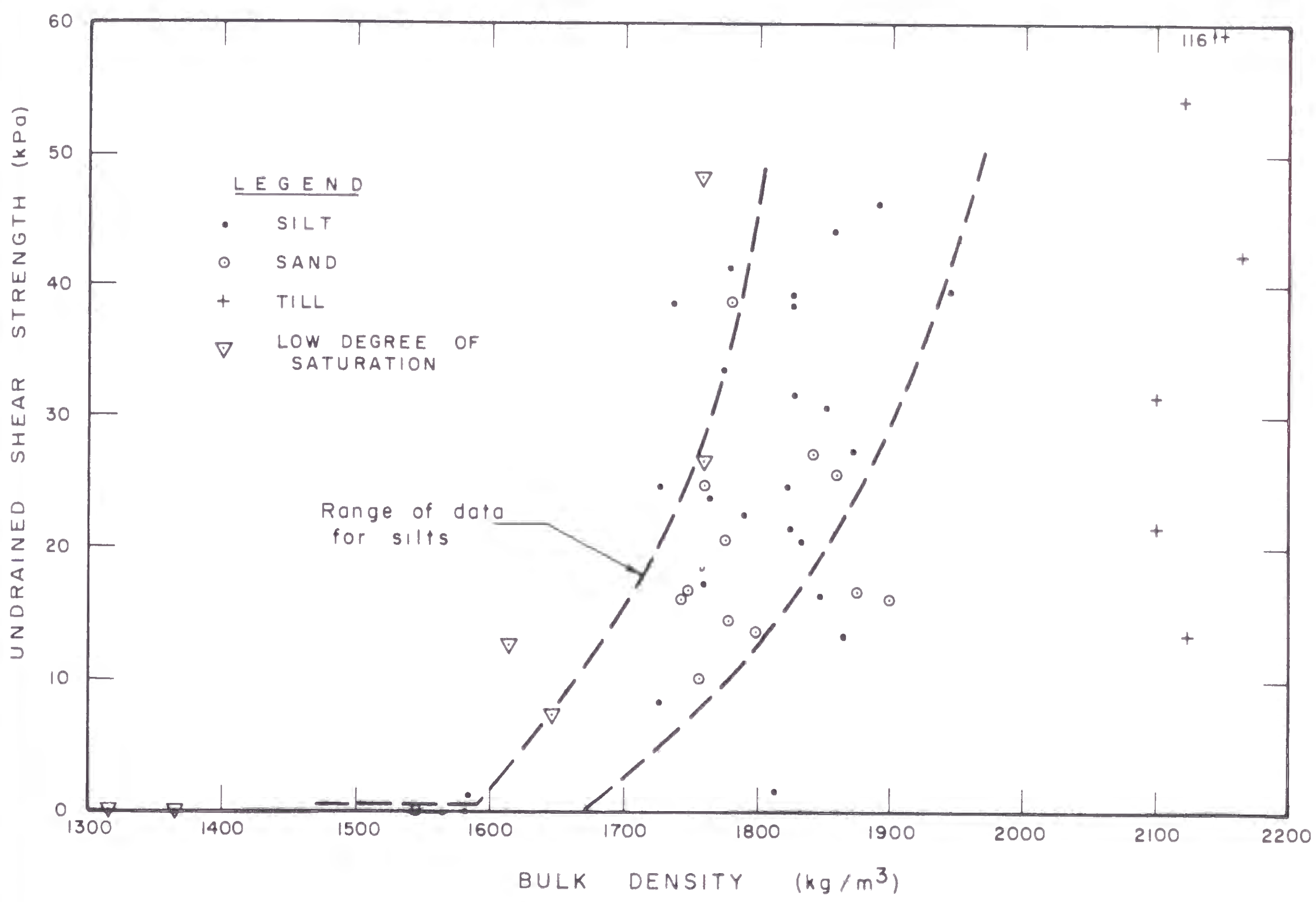


Figure 3.17: Thawed strength (Nixon and Hanna, 1979)

From the above relationship, it can be seen that the thaw consolidation ratio, or the pore pressure, will increase as either the rate of thaw increases or as the coefficient of consolidation (or the permeability) of the soil decreases. Hence the concern for thawing of soils is greatest for finer soils, such as silts and clays which have a lower permeability. The theoretical relationship above indicates that for a given soil, the most effective means of limiting the loss in strength on thawing, is to reduce the rate of thawing.

Thawing and the resulting loss of soil strength is unacceptable for foundations of facilities and hence the design approach must be to prevent thawing. For pipelines, the potential loss of soil strength is of most consequence on sloping ground. In this case the thaw consolidation ratio is incorporated into the conventional stability analysis of the thaw zone beneath the right of way and around the pipeline.

3.3 IMPACT OF DEVELOPMENT ON GEOTHERMAL REGIME

As noted earlier, the mean annual ground temperature within a region tends to be around 2 to 4° C warmer than the mean annual air temperature. Therefore, maps showing mean annual air temperature isotherms can be used to provide a preliminary indication of ground temperatures. Exceptions can be related to extremes in vegetation cover, snow cover, drainage, proximity to water bodies and of course man-made modifications to the surface boundary conditions. Most construction developments in the arctic will tend to increase ground temperatures and the engineer must recognize the impact of this on the integrity of the constructed facility.

Active layer information must be obtained for each specific site, as conditions can vary considerably. It is also important to predict what impact any development will have on the active layer in the long-term. For example, the thickness of the active layer along the south side of a new building will undoubtedly increase as the heat from the sun may be "trapped", together with a probable increased solar reflection off the building. In comparison, the north side of the building will be much cooler.

Any increase in seasonal thaw can have significant impact on many ice-rich deposits. It should be noted that ice is typically most prevalent in the upper 3 to 5 m and often there are distinct layers of ice at the base of the active layer. Extreme thaw settlement can occur as a result of disturbance to the surface. Figure 3.18, illustrates where 60 cm of thaw settlement has occurred on a cleared right of way, relative to the adjacent undisturbed terrain on the right. In this situation, the removal of the trees and damage to the moss and peat cover on the right of way, has caused considerable thaw and settlement. (In the long term, as much as 80 to 100 cm of settlement is predicted.)

Figure 3.18: Thaw settlement on the right of way of the Norman Wells pipeline

As an indication of the impact of surface disturbance or change, it is common to consider the n-factor, which is the ratio of the ground surface temperature to the air temperature. Some typical values for summer and winter n-factors are given in Table 3.1. It is evident how ice-rich terrain might be very sensitive to surface changes. For example, a gravel soil surface exposed by tree clearing (n = 1.3, or greater) would absorb at least three times the summer heat compared to the original treed surface (n = 0.37). This demonstrates the fact that the warm permafrost in the southern, discontinuous zone, will degrade as a result of tree clearing and other surface disturbance from pipeline and facility construction.

It is also possible to cause thermal degradation by construction of gravel pads and by the direct influence of structures. For example, placing a thin layer gravel cover over a natural mossy surface will dramatically increase the amount of heat absorbed by the ground in the summer, similar to the example in the previous paragraph. Likewise, if a structure with a floor temperature of 15 to 20°C is constructed directly on permafrost terrain, the building heat would cause thermal degradation of the ground unless the heat is intercepted by some means. For this reason, most structures, with moderate floor loads, constructed in permafrost regions are elevated 0.5 to 1.0 m above the ground surface, to 'separate' the building heat from the ground.

Table 3.1: Values of n-Factors for Different Surfaces (from Johnston, 1981)

Surface Type	Freezing-n_f	Thawing-n_t
Spruce trees, brush, moss over peat - soil surface	0.29 (under snow)	0.37
As above with trees cleared - soil surface	0.25 (under snow)	0.73
Turf	0.5 (under snow)	1.0
Snow	1.0	-
Gravel (probable range for northern conditions)	0.6 - 1.0 (0.9 - 0.95)	1.3 - 2
Asphalt pavement (probable range for northern conditions)	0.29 - 1.0 or greater (0.9 - 0.95)	1.4 - 2.3
Concrete pavement (probable range for northern conditions)	0.25 - 0.95 (0.7 - 0.9)	1.3 - 2.1

For certain types of development it is neither technically nor economically feasible to prevent the warming and thawing of the ground. The most common example of this situation is large diameter oil and gas pipelines. Insulation is only effective in limiting the impact of moderate amounts of heat input for relatively short term applications. For major heat input, such as from a large diameter pipe at 40°C, insulation is not feasible. However, in cases of warm permafrost, the clearing of the trees for the construction right of way is sufficient to cause thawing of the permafrost. A hot pipeline would cause more rapid thawing, and it is usually necessary to cool the oil or gas to reduce the thawing. In the case of heavy oil pipelines, it may be necessary to maintain a hotter flow temperature and an above grade mode may be required.

3.4 CLIMATE EFFECTS

3.4.1 Normal Climate Fluctuations

It is common to reference the published "Climate Normals" to obtain mean annual air temperatures. These data typically cover a 30 to 50 year period for most major centres. However, there can be considerable variations within the reported period. For example, in Yellowknife, Canada the published 30 year mean annual temperature is -5.4°C. The individual annual mean temperatures within the period range from -3.6 (1953) to -7.0 (1966 and 1972). This range is considered as "normal" climate fluctuations, however, the effect of potential extreme warm or cold years should be considered in design, especially for shallow-founded facilities.

Another means of representing the temperature is by the thawing or freezing indices. These are defined as the number of degree-days of thawing or freezing and are derived by adding all of the positive or negative mean daily temperatures for a one-year period. (The mean monthly temperatures times the number of days in the month can be summed to give the same result.)

For design purposes, it is common to take the average of the three hottest or coldest indices in the last 30 years or alternatively Figure 3.19 can be used to obtain a design extreme index.

3.4.2 Climate Warming

There is evidence that in some areas of the arctic, mean annual air temperatures are gradually increasing whereas in other areas temperatures are constant or even cooling (Lachenbruch *et al.* 1988). Numerous studies have related climate warming to the "greenhouse effect" whereby radiatively active gases (carbon dioxide, methane, nitrous oxide and chlorofluorocarbons) in the atmosphere permit greater penetration of the solar radiation yet prevent the reflected long wave radiation from escaping. It is predicted that even if no more atmospheric pollution occurred, there would still be a rise in temperature due to the present buildup of such gases.

Various predictions have been made (Etkin *et al.* 1988; Etkin, 1989) on the rate of warming that might be anticipated. Three scenarios are presented in Figure 3.20, which shows temperature increases ranging from 0.06°C/decade to 0.8°C/decade. The middle scenario, which assumes current CO_2 emissions and moderate climate sensitivity, appears to give a minimum rate of temperature increase that should be considered in long-term projects. In other words, the mean air temperature may increase by as much as 1°C in the life of most projects. It is considered premature to apply the upper scenario at this stage.

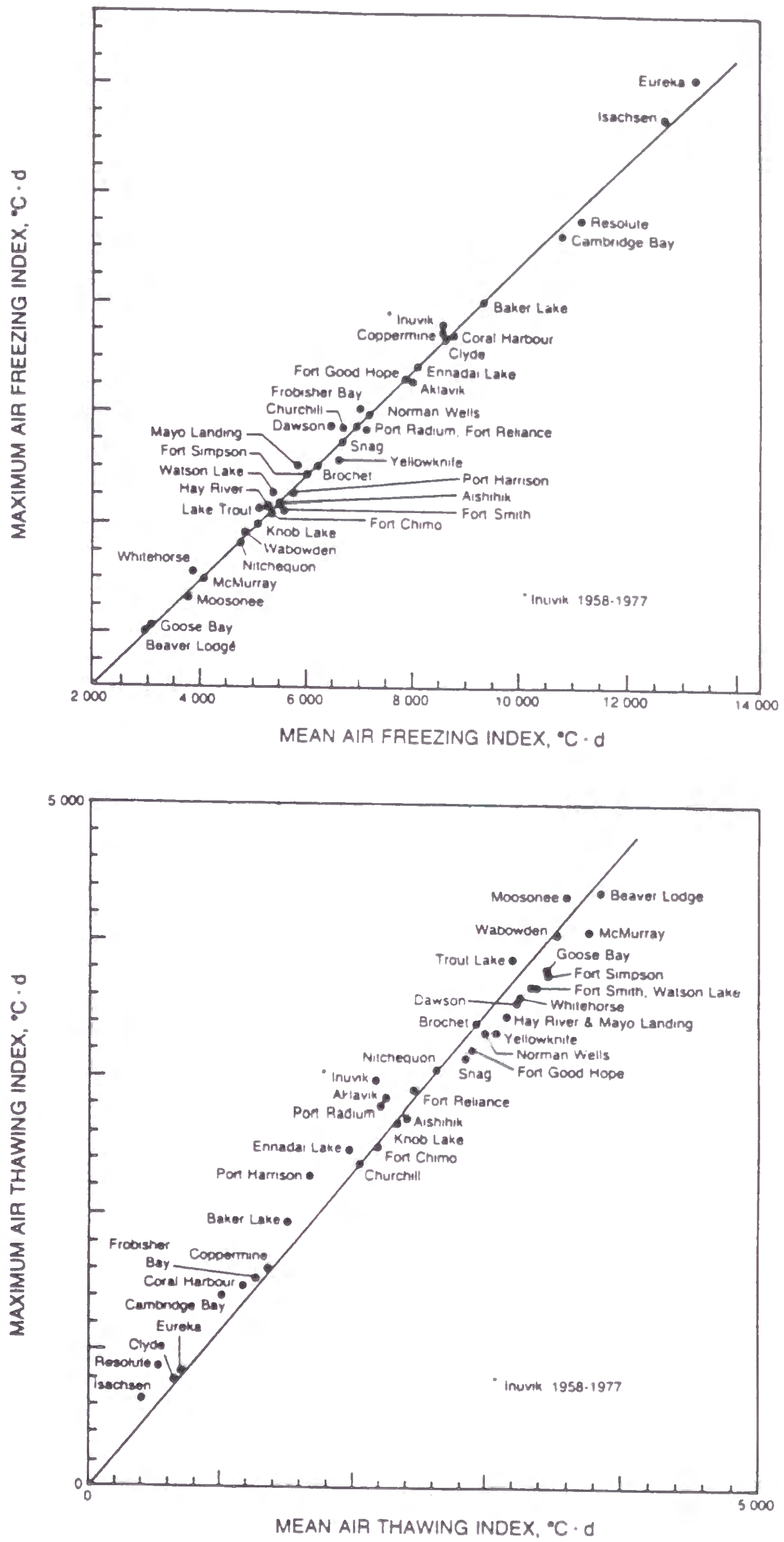


Figure 3.19: Relation between maximum and mean air freezing and thawing indices (in degree days Celsius - DDC) for Northern Canada during the period 1949-1959 (from Johnston, 1981)

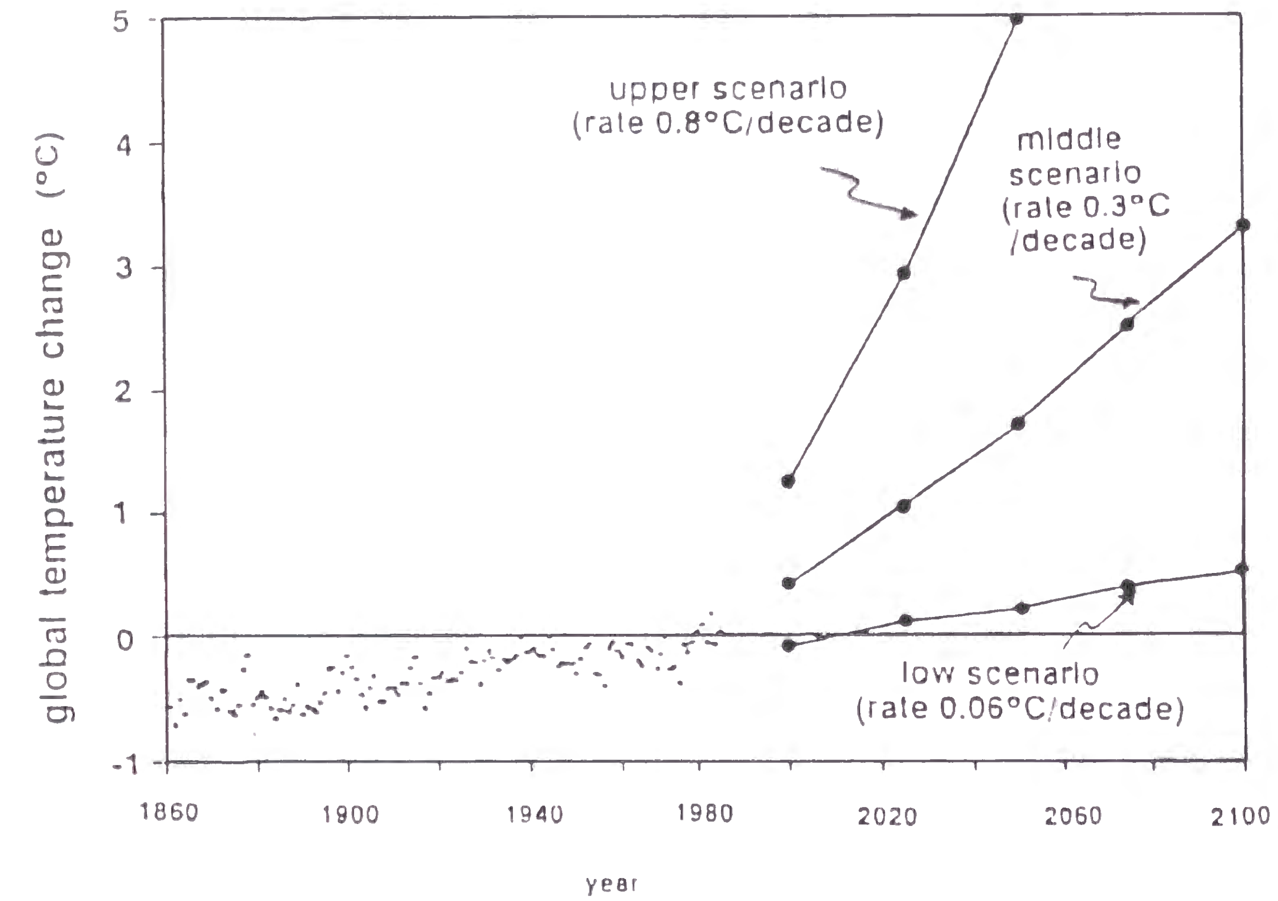


Figure 3.20: Range of probable future global temperature changes (from Etkin, 1989)

Several authors have discussed the impact of warming on permafrost foundations (Esch and Osterkamp, 1990; Nixon, 1990). Both papers examine the impact of the maximum predictions of potential temperature change, 1.0°C/decade, and as such are likely an over-reaction. Nevertheless any warming of the permafrost will tend to cause an increase in creep rates in frozen ground and result in some additional thaw settlement. In the longer term, significant reduction in foundation capacity could occur.

It appears there will undoubtedly be a continued warming in the atmosphere, however, the change should be relatively gradual and the ground, especially at depth, will take some time to respond, Therefore, while the impact on foundations in permafrost must not be ignored, one must resist the tendency to become alarmist. In any event, it should be more economical to add insulation or provide some other remedial measures at a later date, if required, than to over-react now and dramatically limit foundation capacities.

3.5 SITE INVESTIGATIONS

The requirements for any site investigation will depend on several factors, including:

- latitude of site;
- nature of development;
- type of structure (elevated or on-grade);
- magnitude of loads;
- existing geotechnical information (soil, salinity, temperature)
- economic impact of design conservatism. As a minimum for any development, the foundation design engineer should know the soil profile, ice contents, salinity and the mean ground temperature. Generally speaking the higher the latitude and the smaller or lighter the structure, the requirements for detail is reduced. On the other hand, a larger project in the discontinuous or sporadic permafrost zone will require considerably more detail, as maintaining the permafrost will be more difficult and the potential for excessive settlement could be much greater. An outline of a more detailed scope of work for an arctic geotechnical investigation is given in Table 3.2.

In the Russian System, there are specific requirements for the drilling of boreholes and laboratory testing, defined in SNIP 1.02.07-87. The following table provides some information on drilling requirements.

Excerpts from Russian Code for Site Investigations

A. Linear Facilities - Main Pipelines

Borehole Spacing	Pre-Design 500 m, Average	Detail Design 250 m, Average
Borehole Depth	At least 10 to 15 m; 5 m deeper than expected thermal influence.	

B. Buildings and Structures

Ground Conditions	Borehole Spacing	
	Minor Facility	Major Facility
Uniform, continuous Permafrost	40 - 50 m	30 - 40 m
Discontinuous Permafrost	25 - 30 m	20 - 25 m

The typical approach for a long pipeline passing through varied permafrost terrain would be to conduct an airphoto interpretation to map the main terrain units, based mostly on their geomorphic origin. Drilling locations would then be selected so that an adequate number of boreholes were drilled to provide statistical data for each terrain unit. The soil stratigraphy and ice contents can then be generalized for each terrain unit, unless the data indicates a poor statistical set.

On many projects where the permafrost is warm, ice-rich or saline the possibility of founding on competent rock becomes more desirable. Rock profiles can only be determined with accuracy by drilling; however, some geophysical surveys can supplement and reduce the total number of boreholes required.

3.5.1 Airphoto Interpretation

Airphoto interpretation can be used to considerable advantage in the early stages of a project from the perspective of site selection, drainage conditions etc., as well as for obtaining information on the terrain type and potential for shallow bedrock. While examination of airphotos should never take the place of a site visit and proper field investigation, much preliminary planning and evaluation of different site alternatives can be conducted in the office.

With respect to foundation conditions, an experienced geomorphologist can obtain an indication of the types of soil and even likely ice contents. For many permafrost regions of the world, geocryological maps can assist in the interpretation of the airphotos. Mapping of the terrain types is common for large projects, for entire communities in the north or for linear projects such as roads and pipelines. Any such interpretation, however, requires subsequent ground-truthing by drilling.

TABLE 3.2: Scope of Work for Arctic Geotechnical Investigations

1. Review of existing geotechnical/geological information in the region/community (geological reports/maps, other project reports, etc.).

2. Airphoto review of terrain conditions, drainage, existing development/terrain disturbance.

3. Determine:
   - site investigation requirements
   - available equipment
   - mobilization options, if required
   - suitability of geophysical surveys
   - costs of site investigation

4. Conduct site investigation and any geophysical surveys to obtain:
   - stratigraphic detail across the site
   - thickness/condition of existing fill (if applicable)
   - nature and extent of existing surface disturbance
   - indication of thickness of seasonal active layer
   - groundwater conditions in active layer
   - delineation of frozen/unfrozen areas
   - natural ice contents in permafrost
   - ground temperature at depth of zero seasonal amplitude
   - note surface drainage pattern
   - typical foundation systems used for existing structures
   - performance of existing foundation systems
   - availability of granular construction materials (for quality fill, aggregate, etc.)

5. Conduct laboratory tests to obtain:
   - moisture contents
   - confirmation of soil classification (limits, sieves)
   - salinity
   - chemical analysis (if "salinity" not obviously marine)

   Special tests (as applicable)
   - thaw settlement
   - thawed strength (undrained or drained)
   - frost heave

6. Provide in a report, all findings from the investigation and engineering recommendations. Assess all manner of impacts the development will have on ground conditions; provide all feasible foundation systems (including recommendations to consider relevant innovative foundation systems); other site development components such as gravel pads, drainage; geotechnical design review and inspection requirements.

3.5.2 Geophysical Techniques

The geophysical techniques most commonly used in the arctic are electromagnetic (EM) and seismic. In recent years, ground probing radar (GPR) has been developed to the extent that it can be useful in certain circumstances. The following sections provide a brief introduction to the techniques and discusses their value and limitations. In all cases the surveys must be conducted by experienced personnel and should never be relied upon in the absence of boreholes for the confirmation of results.

3.5.2.1 Electromagnetic Surveys

The electromagnetic system consists of transmitter and receiver coils mounted in the ends of a rigid boom 4 m in length, in the case of the EM31, illustrated in Figure 3.21. Current flowing in the transmitter coil generates an electromagnetic field, which in turn causes small electrical currents, called secondary currents, to flow in the ground under the instrument. The strength of these currents depends on the resistivity of the ground. These secondary currents in turn create a secondary electromagnetic field which is measured by the receiver coil in the instrument. The instrument is calibrated to read ground conductivity directly. Electrical resistivity is the reciprocal of electrical conductivity and so can be easily calculated.

The operating frequency of the instrument is chosen so that the depth of measurement is controlled by the geometry of transmitter and receiver coils. When the plane of the coils is horizontal, penetration is about 7 m below the instrument; when the plane of the coils is vertical, penetration is reduced by one half. By taking readings in both coil positions it is possible to determine whether the conductivity increases or decreases with depth.

Readings are typically taken at 5 to 10 m intervals along a line. By plotting the results it is possible to note changes in terrain signals. With the assistance of some borehole information, the experienced interpreter can delineate zones of frozen or unfrozen ground and to some extent coarse or fine-grained soils. Electromagnetic surveys have been used very successfully on arctic pipeline projects in the discontinuous permafrost zone to identify the frozen/unfrozen interfaces (e.g. Kay *et* al. 1983) and in sub-arctic regions to detect isolated permafrost zones.

3.5.2.2 <u>Ground Probing Radar</u>

There have been tremendous advances in the development of ground probing radar capabilities for subsurface surveys in recent years. State-of-the-art equipment has been developed and is known as the Pulse EKKO IV system. Some features and applications are shown on Figure 3.22. Surveys have been successfully conducted for a wide range of applications including; permafrost, bedrock (profile and structure), buried objects (pipes, tanks, etc.) and stratigraphic profiling. In permafrost applications, GPR is capable of identifying frozen/unfrozen interfaces and subsurface massive ice bodies (Dallimore and Wolfe, 1988).

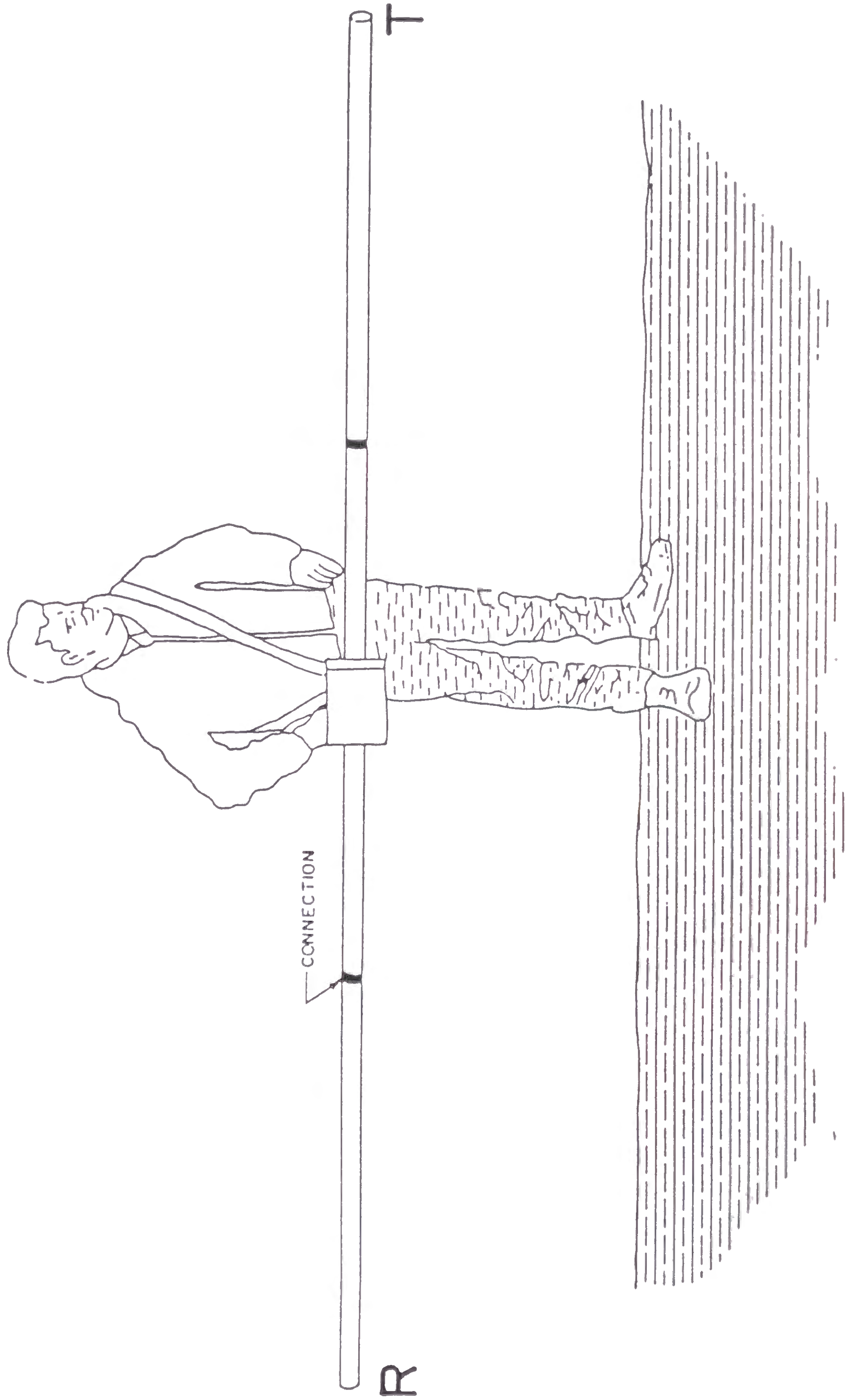


Figure 3.21: Inductive conductivity-measuring systems, schematic illustration

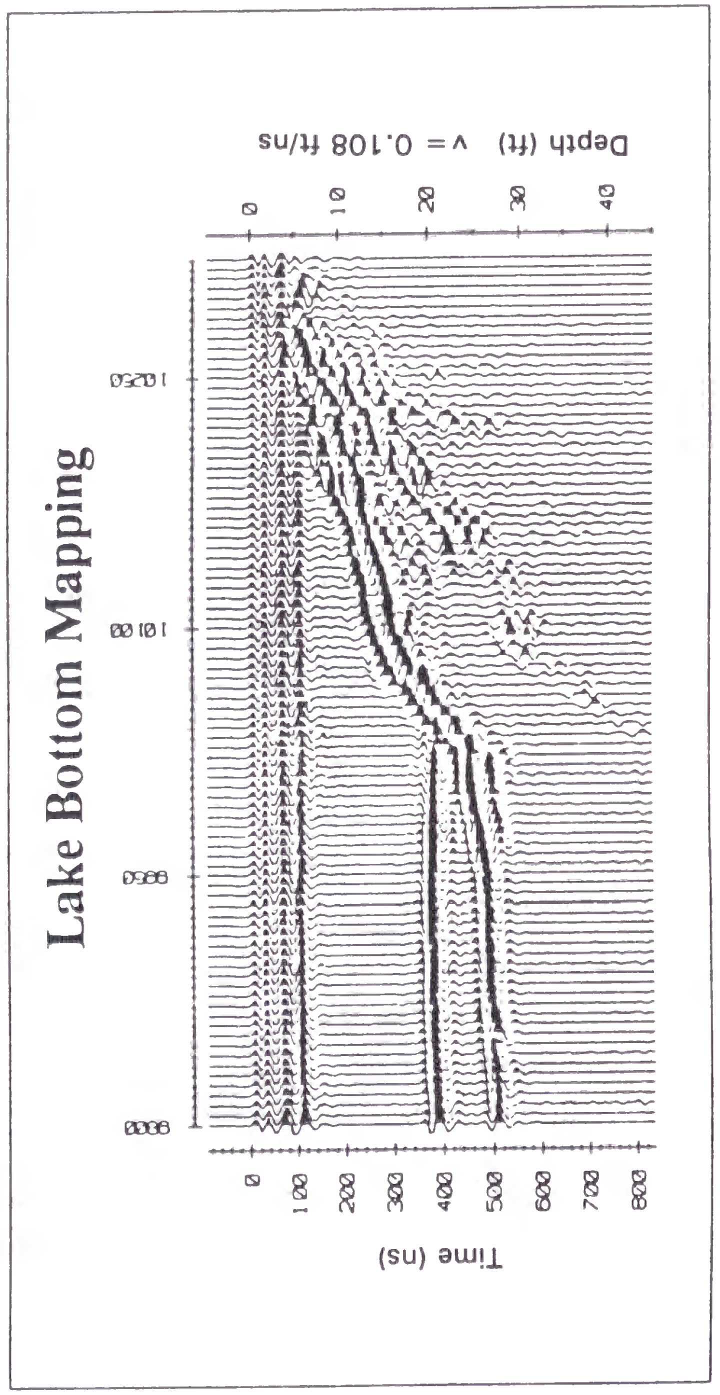


Figure 3.22: Ground probing radar applications

3.5.2.3 Seismic Refraction

Relatively shallow seismic refraction surveys can be conducted quite simply on the ground surface. The seismic wave is generated by striking a metal plate on the surface, The refracted wave is then received by a geophone connected to a seismograph.

In the field, the typical procedure would be to establish the geophone at one end of a 30 m long profile, and then to hammer the metal plate at successive 1 m intervals to the other end of the profile (Figure 3.23a). For each station, the time taken for the signal to travel from the hammer to the geophone is recorded. Finally, the geophone is moved to the other end of the profile and the hammering process repeated back to the starting (i.e. original geophone) position.

A typical plot of travel times as a function of hammer-geophone distance, obtained as described in the previous section, is shown on Figure 3.23b. The travel time curve can be broken into straight-line segments, each of which may be correlated with a layer of varying seismic velocity in the subsurface. The inverse slopes of the straight-line segments give the velocities of the appropriate layers.

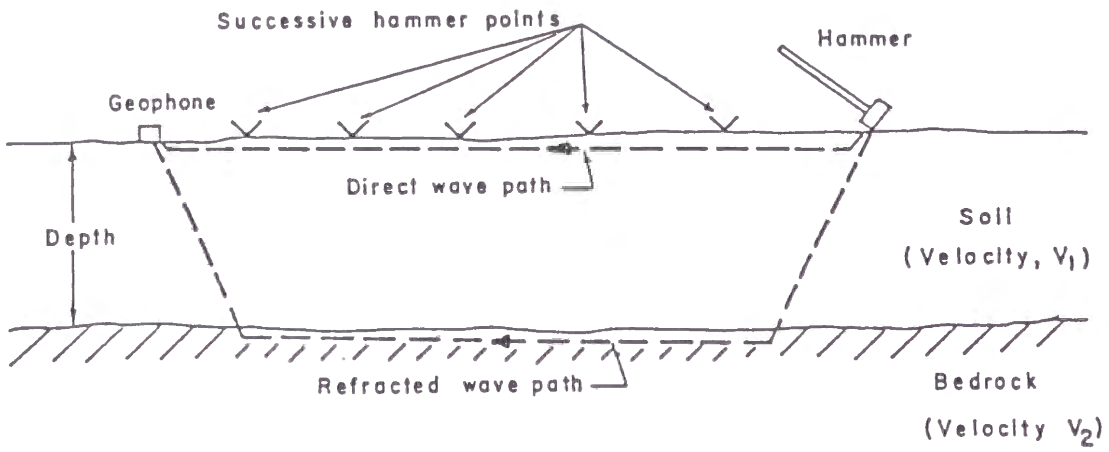
Depths to the interfaces between layers are calculated from the velocities and either the intercept times of the segments or the critical distances for which travel times are equal through two adjacent layers (ti or Xc respectively on Figure 3.23b). These interpretation procedures assume that subsurface layers have approximately planar interfaces and that seismic velocities increase with depth.

3.5.3 Drilling Equipment

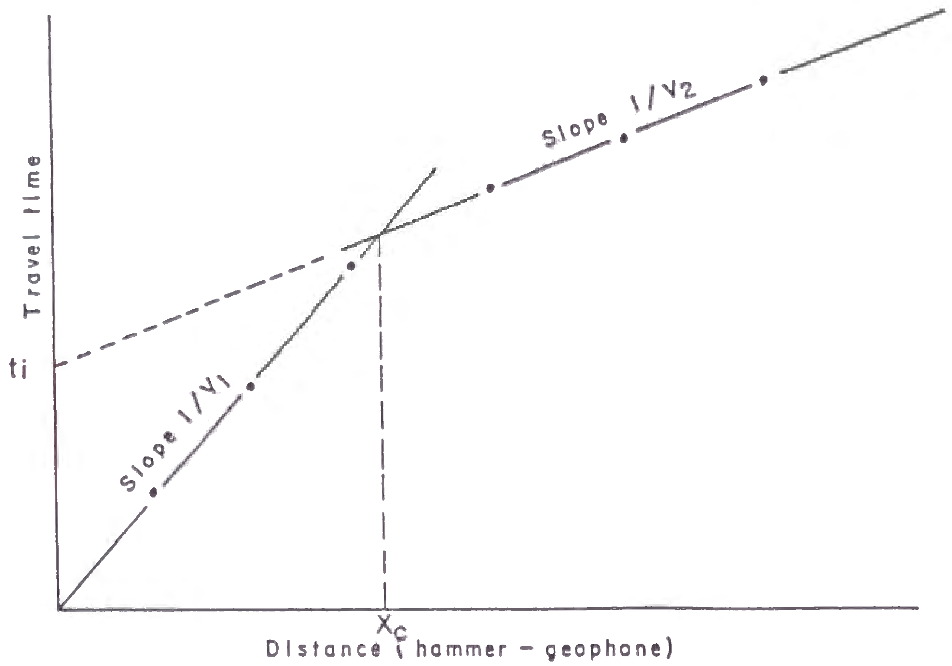
3.5.3.1 Conventional Equipment

There is sometimes only a limited amount of geotechnical drilling equipment that is based in the arctic. A recent review of drilling and sampling in Alaskan permafrost has been presented by Riddle and Hardcastle (1991). It should be appreciated that test pits (manual or backhoe) or hand-augering are a possible method of investigation for very remote sites, however, the extent of investigation is usually limited to the active layer. For most foundation designs it is important to have data on the ground conditions beneath the active layer. The most common equipment used includes drills mounted on trucks, tracks, balloon tires and skids. Some are primarily auger drills; others are rotary drills that use air or water as a circulation medium. Some of the smaller units are readily transportable by helicopter, though usually in several components

Certain drills are better suited to more difficult drilling conditions as outlined in Table 3.3. The method of sampling varies between soil conditions and drill type as also shown in Table 3.3. Where possible, it is desirable to obtain a "core" of the permafrost to enable a good visual logging of the ice conditions.



a) SURVEY GEOMETRY



b) TRAVEL - TIME PLOT

Figure 3.23: Shallow refraction seismic survey

Table 3.3: Drilling in Arctic Regions

Soil			Rock	
Fine-Grained		Granular	Sedimentary	Metamorphic and Igneous
Warmer than -5°C	Colder than -5°C			
Drilling Equipment (typical maximum depth, m)				
Auger (30) Rotary (100) Sonic* (30)	Rotary (100) Sonic (30) Airtrack (50) Diamond (100)	Rotary (100) Sonic (30) Hammer (100) Airtrack (50)	Rotary (100) Airtrack (50) Diamond (100)	Diamond (100) Airtrack (50)
Sampling Methods				
Modified CRREL** barrel auger sample split spoon cuttings	as for warm soil; sonic drill produces continuous disturbed sample	as for fine soil; sonic and hammer drills produce pulverized samples	cuttings; core barrels (e.g. VTM, Christensen, (diamond)	cuttings; diamond core barrel

Notes:

* A sonic drill relies on a hydraulically powered oscillatory drive unit (or tub) to create resonance in the drill pipe. Penetration is possible through boulders and coarse deposits. Though much thermal disturbance is caused, a good stratigraphic log and reasonable moisture data can be obtained.

** Cold Regions Research and Engineering Laboratories (U.S. Army).

3.5.3.2 Helicopter Drills

Several types of helicopter drills have been used for remote, linear projects over the past 20 years. All forms of drilling and sampling summarized in Table 3.3 are available in helicopter configuration except the hammer drilling. Due to the helicopter support required, this type of drilling is very expensive and only used in special circumstances,

3.5.4 Ground Temperature Monitoring

The ground temperature experiences considerable seasonal fluctuations within the upper soil profile, to about 3 m depth. The depth at which there is a zero amplitude in seasonal temperatures is in the order of 10 to 15 m depending on the ground surface conditions and the amplitude of the air temperatures. For most foundation designs on permafrost, the ground temperature conditions are important. Ideally the ground temperature should be measured at depths of 10 to 15 m. If a temperature sensor can be installed that deep, the mean ground temperature can be obtained at any time of the year. For shallower installations, the temperature must be recorded at several evenly distributed times through the year in order to obtain the mean.

In remote locations it is not always possible to have readings obtained regularly and data loggers should be considered. Most loggers can be disconnected from the temperature cable and shipped to home base for down-loading and then be returned to the site for re-connection. Alternatively by means of a modem, data can be retrieved by phone enquiry.

In order to confirm the depth of the active layer, it is necessary to have relatively closely spaced thermistors in the expected depth range and to obtain readings at weekly intervals in the fall. The further north the site, the earlier and faster will be the onset of freezeback and timing may be crucial if manual readings are being obtained.

3.6 LABORATORY TESTING REQUIREMENTS

Sample preservation can be critical if undisturbed frozen cores are required for special laboratory testing. In the field, portable propane freezers can be used until the samples can be transferred to a regular freezer at the base of operation. Shipment to the testing laboratory is usually in heavily insulated boxes packed as full as possible with core. Dry ice is used to assist in keeping the samples frozen.

All investigations should include the routine soil classification and moisture content tests, If frozen core is obtained, frozen bulk densities should be obtained to give a further indication of the amount of excess ice.

Of particular importance in permafrost investigations is the salinity test (Hivon and Sego, 1991). Salinity in the porewater can reduce the freezing point and cause dramatic reduction in the strength-deformation properties of the frozen soil. Most salinity is in the form of sodium chloride from the marine environment, however, other salt forms due to regional bedrock influences or river flooding/evaporation cycles are also possible.

The most common laboratory testing for pipelines in permafrost is the thaw settlement test. This test has been developed to quantify the amount of settlement that would occur under a certain load, due to settlement and consolidation as the ice in the sample melts and drains out of the pores. A typical test procedure is described in Hanna et al, 1983, as well as statistical data from a large number of tests for various soil types. Other data, for more coarse grained soils is presented by Nelson *et al.* (1983).

Other tests for pipelines in permafrost could include frost heave tests. However, the preferred pipeline design approach is to avoid or prevent frost heave.

There is a requirement to determine the <u>thawed</u> strength of soils encountered on frozen slopes along the pipeline. These tests will be conducted as traditional unfrozen triaxial tests, to obtain the apparent cohesion and the angle of internal friction.

For very major projects, strength and deformation testing of permafrost soils may be conducted for input to the design of foundations in permafrost. However, there is a wealth of relevant data available in the literature that should be sufficient in most cases.

4.0 GENERAL ENGINEERING ASPECTS OF PIPELINE CONSTRUCTION IN PERMAFROST - DESIGN CONCERNS AND APPROACHES

4.1 GENERAL DESIGN APPROACH AND PHILOSOPHY

4.1.1 Design Approach

For any project, the primary design philosophy is to develop a design that accommodates and complies with the design data, criteria, and codes and which has a minimum overall life-cycle cost. This approach will produce a lower cost of transportation and improve the economic viability of the proposed pipeline transportation system. The author has learned that a key strategy in achieving this objective is to **adopt a design for permafrost that permits burying the pipeline** as much as possible. The primary burial strategy will be supported with secondary strategies that also contribute to achieving a low life-cycle cost, such as minimizing the construction period and optimizing the compressor stations.

Pipeline systems have traditionally been designed to minimize either the initial capital cost or the overall life-cycle cost using an appropriate discount factor. The buried pipeline design results in the two criteria being congruent: minimizing initial capital cost also minimizes overall life-cycle cost. In fact, the idea can be taken further - initial capital cost can be significantly reduced by a willingness to incur somewhat higher operational costs for monitoring and maintenance. The increased operational costs are associated with ensuring integrity of the pipeline system and avoiding adverse environmental impact from phenomena such as thaw settlement of the buried pipeline or thawing slope instability in the permafrost areas. Adopting this approach can avoid the alternative above-ground design for permafrost, with its relatively high capital costs.

This primary strategy has implications not only for the project proponents and users but also requires special consideration in the regulatory process, such as:

1. A strain-based plastic design criterion for buried pipelines in permafrost, as accepted in North American codes and practice.

2. Monitoring and interventive maintenance during the operational period to preserve and restore pipeline integrity and minimize environmental impact.

In developing the conceptual designs the following principles are used:

1. Ensuring designs are flexible and robust to cover a fairly wide range of conditions, rather than fine-tuned to a specific scenario.

2. Giving high weight to constructability and logistics in developing designs. For instance, most pipeline contractors equipment is adapted for and their personnel are experienced with buried pipeline construction rather than large-scale installation of piles and support systems.

4.1.2 Design Codes and Standards

The design, construction and operation of the pipeline system would normally be governed by the national codes regarding pipelines and the many subsidiary national codes. The American pipeline design code, ASME B31.8 is widely used for international pipelines. This code is recommended since:

1. The code has wide international acceptance and it is relatively easy to procure material and services that will conform to it.

2. The code contains explicit permission for plastic design criteria that is a central concept in burying pipelines in permafrost.

4.2 UNIQUE PERMAFROST DESIGN ISSUES

In this section the author presents an overview of the design concerns and the design approaches that are unique for pipelines in permafrost regions as opposed to conventional pipeline practice in more temperate climatic regions. The mitigative designs required to address these issues are introduced. The specific design and application of the mitigative designs for the pipeline being studied will be developed in Section 5.

The major issues that require special attention for pipeline design in permafrost regions include:

1. Thaw settlement
2. Frost Heave
3. Pipe structural integrity
4. Muskeg and swamps
5. Slope stability
6. Foundations for Facilities
7. Drainage and erosion problems
8. Cold regions operations problems

The remoteness of the northern permafrost regions can present other special problems for pipelines, including:

1. Long distances to major gas markets
2. More difficult construction conditions
3. Less developed infrastructure for construction and operations.

There are often other difficulties for remote regions, such as more limited geotechnical and other data. The costs of obtaining adequate data can be considerable. Another major impact of the remoteness of northern pipelines is that often the special permafrost designs, and difficult and costly logistics and construction, tend to make the economics of such pipelines marginal. Overly conservative designs may result in pipelines that are technically but not economically feasible. Designs that are inadequate for the extreme and sensitive conditions, may only result in major operations disruptions or a short project life. Innovative and realistic designs are required so that pipeline projects in permafrost regions can be viable.

4.3 DESIGN CONCERNS AND APPROACHES

The most significant aspect of pipeline design unique to permafrost regions is maintaining the structural integrity of the pipeline. The threat to pipeline integrity arises primarily from potential mechanical instability of the surrounding soil. These soil movements are usually attributable to changes in the thermal stability of the soil arising from phenomena such as thaw settlement, frost heave or slope movement, or to secondary effects such as erosion from water movement after soils thaw.

4.3.1 Thaw Settlement

Thaw settlement results when permafrost containing excess ice thaws and the water drains out of the soil mass. The soil consolidates as a result of this loss of volume occupied by the excess ice, resulting in settlement of the soil mass. Thaw settlement can arise:

1. Naturally, from causes completely unrelated to pipelines;

2. Indirectly, as a result of pipeline related activities (such as after clearing of vegetation from the right of way)

3. Directly, from the thermal influence of the heat flux from the pipeline.

Natural thaw settlement occurs in areas where vegetation cover may have changed as a result of extreme or gradual climate or drainage conditions. More extreme changes often occur as a result of natural forest fires. If the active layer increases, for a variety of natural reasons, ice rich soils typically existing in the upper permafrost layers, will thaw and settle. Sometimes this can then lead to a gradual process whereby, water collecting or flowing on the surface then tends to promote further thawing in subsequent seasons. This can lead to “drunken forests”, or in extreme cases, to thermokarst lakes.

In discontinuous permafrost regions, the clearing of the trees for the pipeline right of way can alter the surface heat balance sufficiently to initiate permafrost degradation and thaw settlement. This results from the increased solar radiation on the ground surface. The original tree cover provided shade and in many treed areas, there would also be a thick cover of moss. The direct sunlight would usually dry and kill the moss, which provided a good insulating surface. Hence, the ground surface temperature becomes several degrees warmer than before. In the discontinuous permafrost region, the permafrost is no colder than about -2°C, therefore long term permafrost degradation will result.

The additional influence of a pipeline installed in the right of way depends on the operating temperature, the diameter, the contents and the flow. The larger the diameter the greater the influence. A pipeline less than about 18 to 24" in diameter will have a limited thermal influence near the station outlet, however, heat loss to the ground eventually reduces to a minimal value. On the other hand, a large diameter pipeline can have a significant additional thermal influence, especially an oil pipeline.

The outlet temperature from a compressor station on a gas pipeline could be in the order of 40 to 60°C, if permitted. However, temperatures above 40°C may be harmful to certain types of pipe coating and to the overlying vegetation or crops. Hence, even in southern gas pipelines, some level of cooling is usually required. If a large diameter gas pipeline was allowed to operate as warm as 40°C in permafrost, there would be considerable thaw around the pipeline. Depending on the ice contents, this thaw would create a significant amount of thaw settlement. The most critical aspect for the structural integrity of the pipe is the differential settlement that can arise because of several factors:

1. Transitions between unfrozen and frozen terrain

2. Transitions between different geomorphic terrain units, with different ice contents, and

3. Natural variations in thaw penetration and ice content within a given geomorphic terrain unit.

One approach to design the pipeline for such situations, would be to predict where such transitions occur. It might be possible, in the first two situations noted above, to predict transitions with some degree of confidence, however, in practice it is often found that there are very many transitions. The third situation is very difficult to predict. Therefore, it is usually concluded that all of the pipeline must be designed for the maximum likely differential thaw settlement. The main means to limiting thaw settlement will be to avoid terrain with significant ice content if possible and to limit the pipeline temperatures.

4.3.2 Pipeline Operating Temperatures

The desirable operating temperature for gas pipelines in permafrost depends primarily on how extensive and cold the permafrost is. As a general rule, gas pipelines will be operated as a "chilled" pipeline in the continuous permafrost region. This is based on the fact that permafrost exists everywhere except beneath any significant creeks and rivers. Therefore, thaw settlement can be avoided by chilling the gas to operate at -5°C or colder. The design for the thaw zones beneath rivers is to provide significant insulation and heat tracing to minimize any frost penetration.

In the discontinuous permafrost region, the unfrozen terrain becomes more and more prevalent towards the south and hence chilled flow is no longer feasible. For north - south pipelines, starting in the continuous permafrost, it is necessary to determine the "last point of cold (chilled) flow". South of this point, the pipeline will be operated as a "warm" pipeline and thaw settlement will be the main design issue. In order to minimize the amount of thaw settlement it may be necessary to operate the pipeline as a "cooled" pipeline. Such pipelines would normally be operated in the range of 0 to 15°C, requiring active cooling using aerial or refrigeration coolers.

The current North American approach to the structural design of gas pipelines in discontinuous permafrost is:

1. Select the pipeline route to avoid high ice-content terrain as much as is practicable;

2. To reduce the expected thaw settlement by reducing the pipeline operating temperature, and;

3. To design the pipe to withstand significant differential settlement, by allowing the pipe to experience plastic strain (discussed in more detail in Section 4.3.4) and adopt a sophisticated pipeline deformation monitoring program so that interventive action can be taken when and where required.

In some regions, route selection can only be of a limited benefit with respect to the thaw settlement concern. Often, the ice rich terrain is quite extensive and can not be avoided. Even if the ice rich conditions are intermittent, attempts to route around such areas will be limited by the fact that any overall lengthening of the pipeline has considerable cost implications.

The effectiveness of the reduced operating temperatures is illustrated on Figure 4.1. The thaw beneath the pipe is significantly reduced for the 10°C case compared to the more normal 40°C case. The design objective is to determine the optimum maximum station outlet temperature that will result in a predicted thaw settlement that will be tolerable for the pipe structural design. There may be several iterations involved in this pipe-soil interaction design process. For example, the mechanical design for the pipe will dictate a certain wall thickness, which will therefore have a related structural capacity to withstand a certain amount of differential settlement. It is necessary to determine the optimum design by either reducing the thaw settlement by further reduction of operating temperatures versus increasing the pipe wall thickness so that more thaw settlement can be tolerated.

There are other options or alternative design/construction modes available to minimize thaw settlement, mostly with significant cost increments. These will be discussed in Section 4.3.5.

The reduction in the gas temperatures requires cooling following compression at each station. A limited amount of cooling can be achieved with aerial cooling units. However, for cooling to 10°C more active refrigeration units are required.

In the discontinuous permafrost region, it is also necessary to limit the minimum temperature at the inlet of the compressor station, due to the potential for frost heave. The Joule-Thomson effect causes considerable cooling of the gas as the pressure reduces along the pipeline. If the gas is flowing at a maximum temperature of (say) 10°C, then the gas will cool to 0°C in a much shorter length of pipe than if the gas was allowed to flow at 40°C. Hence, for a cooler pipeline, closer station spacing is required to reduce the risk of frost heave.

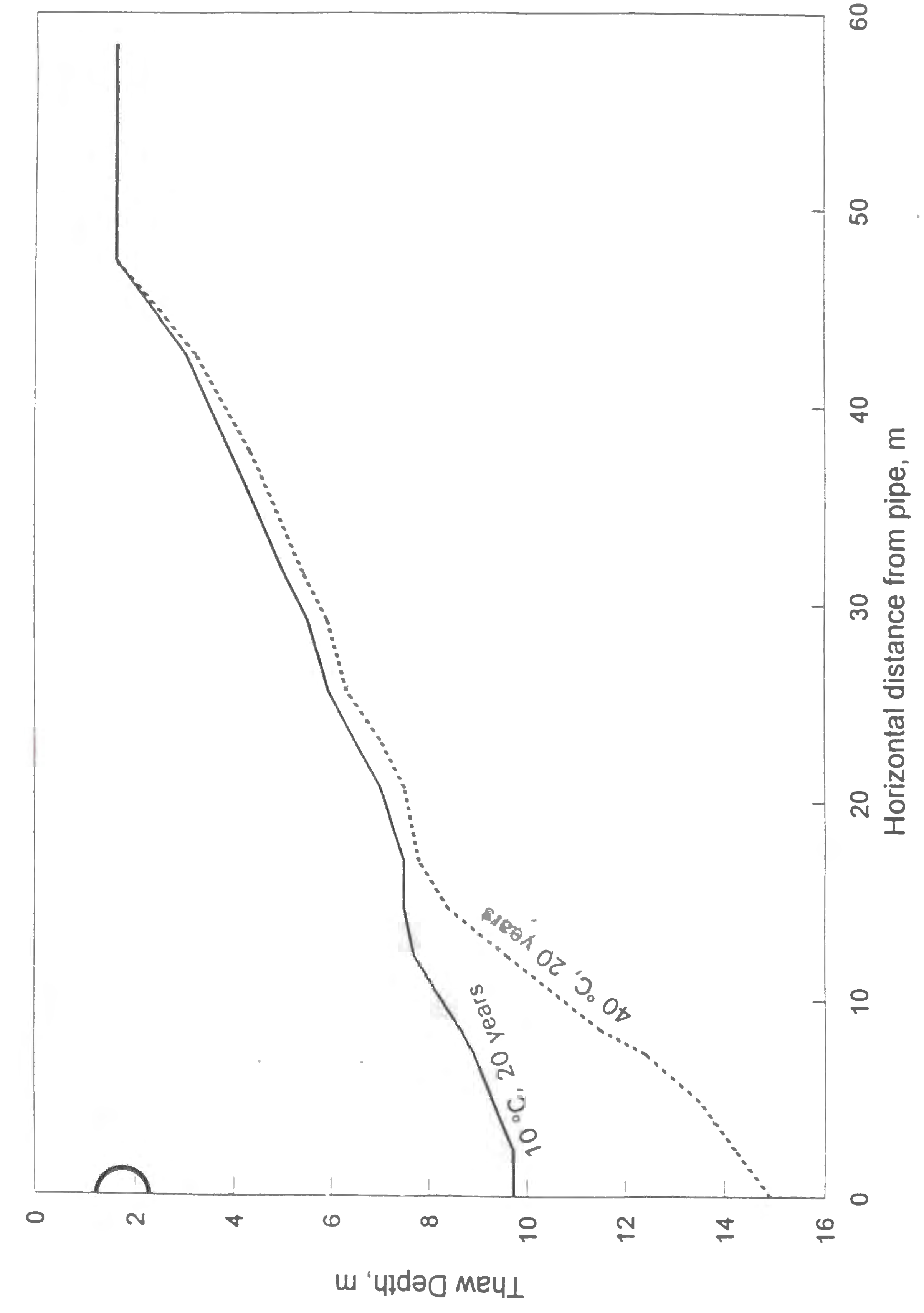


Figure 4.1: Thaw progression under pipe and adjacent area

4.3.3 Frost Heave

It has been noted that for pipelines in the colder, continuous permafrost, the gas would normally be chilled to -5^{0}C following compression. Pressure loss will result in further cooling along the pipeline. Under these cold operating, there is a risk of frost heave developing in unfrozen zones associated with rivers and streams. Since these areas are easily predictable, the design for frost heave mitigation can be applied to the specific required pipeline sections. The existence of other unfrozen or partially frozen zones existing along the route can be verified using geophysical survey techniques. Frost heave occurs as ice lenses form during the freezing of wet soils containing significant silt or clay particles (Penner 1960, Konrad and Morgenstern 1980, and Nakano and Takeda 1994). The prediction of the extent and rate of frost heave is not very reliable. The forces involved during heaving are considerably greater than those involved with thaw settlement and hence it is not considered advisable to attempt to design the pipe to withstand frost heave. As such, rather than design for frost heave, the approach is to prevent it from occurring in the isolated unfrozen areas, by use of insulation and heat input to counteract the freezing temperatures. Above-ground pipeline sections are another alternative.

For a "cool" pipeline in discontinuous permafrost, there is a potential for frost heave to occur near the inlet to compressor stations. The selection of the location of the compressor stations will be primarily influenced by the need to minimize the risk of frost heave occurring in the winter months. Again the approach will be to avoid the occurrence of frost heave, by limiting the distance between stations.

4.3.4 Pipeline Structural Integrity

Stress analysis is concerned with the loadings on the pipeline and the required designs to resist them adequately and ensure the integrity of the pipeline. The categories of loads that a pipeline may be exposed to may be broadly categorized as:

1. Loads from internal pressure
2. Loads imposed during construction.
3. Loads imposed during operation

For most conventional pipelines operating at moderate temperatures, only the first category is significant and is routinely solved by Barlow's formula without need for specialist stress analysis. The other categories of load usually require only a quick check to determine if they apply. Stress analysis, if used at all, is reserved for special situations such as compressor station discharge piping.

For pipelines in permafrost the third category of load, operational loads, is very significant, and may be the determining factor in design. In permafrost regions, loads will be imposed on the pipe as thaw settlement (or frost heave) proceeds. The pipeline must resist loads in two modes:

1. As a pressure vessel
2. As a beam

Stress design for pipelines is performed in according to the governing design codes. Historically, these have applied uniaxial and triaxial stress failure criteria with a suitable margin below the yield strength of the steel. The ASME B31.8 code is taken as the basis for the present preliminary design.

Recently, the codes have made explicit allowance for more sophisticated stress analysis and specifically for the application of plastic strain design criteria to secondary loads. Much of the impetus for strain design criteria has come from offshore pipeline technology, where many problems of "beam support" of a pipeline occur; some of these conditions are analogous or comparable to the beam support problems in permafrost.

Plastic strain criteria are applicable to secondary loads where fatigue (e.g., from cyclic loading) is not a factor. Primary loads are those that would continue to produce displacements indefinitely after yield, such as internal pressure. Secondary loads are by definition inherently self-limiting with respect to the ultimate displacement that would occur, such as loads from thermal expansion.

A detailed rationale and justification for the application of limit-state design using strain criteria for secondary loads, such as thaw settlement, are presented in Appendix A. Recommended preliminary values of the strain criteria are also provided. Some key points are:

1. The ASME B31.8 code presents detailed stress criteria that may be regarded as the customary basis for conventional design

2. The ASME B31.8 code explicitly permits design based on sophisticated stress analysis and limit state design based on strain criteria. It does not however provide a detailed design basis for application of strain-based design. Other codes, such as the ASME Boiler and Pressure Vessel Code provide some guidance in these matters.

3. Specific strain criteria for pipeline design must be enunciated since the pipeline codes provide permission, but do not supply a design basis. Representative values are provided in Appendix A, of three categories of strain criteria for various secondary loadings and combinations of primary and secondary loadings that could be used for preliminary design:

    - Tensile strain criteria
    - Compressive strain criteria
    - Ovalization criteria

Pipeline designs for permafrost regions must be performed according to the local pipeline and design codes. Where these codes do not contain explicit permission for plastic design, a significant effort may be required to convince local technical agencies and regulatory authorities to grant permission to use strain criteria for this pipeline project. These efforts may require a lengthy schedule of consultations and possibly even regulatory or statutory hearings and legislative amendments.

4.3.5 Alternate Design Approaches and Design Modes

The preceding discussion has assumed that the pipeline would be constructed in a conventional buried mode, that is, with about 0.7 to 1.0 m of normal soil cover. Some other design approaches and construction modes that have also been considered and used in North America and Russia are illustrated on Figure 4.2. The various alternatives include:

1. Route the pipeline to minimize exposure to the soil phenomena (thaw settlement or frost heave);

2. Allow indirect displacements, such as thaw settlement, to proceed for some time before construction by, for instance, pre-clearing the right of way well in advance of pipeline construction;

3. Manage the thermal disturbance to the soil and the ensuing soil phenomena by controlling pipeline flowing temperature;

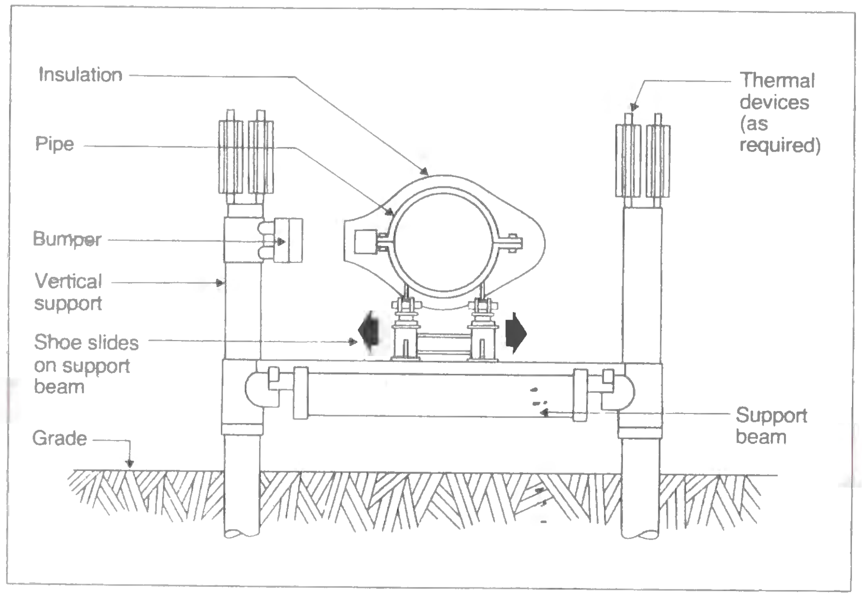
4. Separate the pipeline thermal environment from the soil thermal environment by, for instance:
    - insulation of the pipe or ditch
    - construction of the pipeline above-grade on elevated supports
    - installing the pipeline in an above-grade berm

5. Modify the soil environment around the pipeline by, for instance, overexcavation and use of select granular backfill

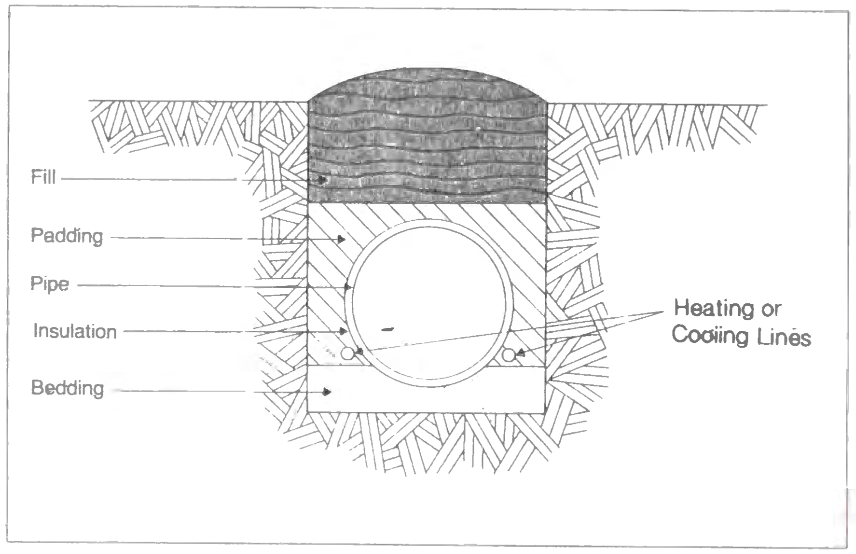
6. Design the pipeline to withstand the full extent of soil displacements

7. Design the pipeline to withstand moderate soil displacements in conjunction with monitoring and interventive maintenance to restore and preserve pipeline integrity

It is understood that on a long-distance pipeline through continuous and discontinuous permafrost regions, it is likely that most of the foregoing strategies would be applied. The key determinant of overall pipeline cost is which techniques are used on a general basis and which are applied as special solutions for localized problems. A most effective combination of strategies for general design in both continuous and discontinuous permafrost, is to use Approach 7 (design the pipeline to withstand moderate soil displacements together with monitoring and interventive maintenance) in conjunction with Approach 3 (manage the thermal disturbance to the soil and the ensuing soil phenomena by controlling pipeline flowing temperature).



a) Alyeska pipeline above-ground design.



b) Buried pipe with cooling or heating lines depending on pipe temperature and ground conditions.

Figure 4.2: Alternate design and construction modes

Some of the above strategies have a number of subsidiary issues and approaches associated with them. For instance, some design questions associated with above-grade construction of a gas pipeline on supports include:

1. The type of support (e.g., single pile, "T" support, versus double pile, "inverted U" support);

2. The size of pile support and the installation method. For instance, piles of about 125 mm diameter or less can be drilled with conventional air drills used for rock with controlled distance between the piles. This method requires many small piles but uses equipment and methods familiar to most pipeline contractors and does not have inherent delays while waiting for freezeback. There are several other pile installation techniques, such as oversized holes with slurry freezeback;

3. The plan layout to permit expansion and contraction (e.g., zig-zag versus periodic expansion loops);

4. The sliding pad arrangements for movement of the pipeline on the supports;

5. The anchoring arrangements to prevent excessive movement of the pipeline on the supports after several thermal cycles, i.e., "walking of the pipeline." For instance, a common design for a zig-zag layout would be a four-pile anchor at the midpoint of each straight section. Some anchor designs include inclined piles;

6. The fracture initiation and arresting design features. Uninsulated pipelines above grade can be exposed to very low ambient temperatures. Moreover, an additional allowance must be made for the Joule-Thomson cooling that would occur during a depressurization event;

7. The type of thermo-piles, if required, and their installation. Thermo-piles provide passive soil refrigeration using single-phase fluid convection or two-phase heat-pipe convection with gas-liquid phase change. The thermo-piles can be separate or integrated with the pipeline support piles;

8. The spacing of support piles may be not determined by operating conditions. If pressure testing with water is used, the great weight of the test water may determine support spacing. Alternatively another test method, such as air, may have to be used which has implications with respect to the test pressures that may be used;

9. There may be other considerations, such as animal passage (e.g., ungulate migration) or vulnerability to vandalism (e.g., rifle shots).

4.3.6 Slope Stability

The proposed pipeline can be expected to traverse many slopes associated with numerous river and creek crossings. Since these slopes in the discontinuous permafrost will be relatively warm, they will be particularly susceptible to thawing. Clearing the trees and disturbing the moss cover will result in significant ground surface warming and thawing of the permafrost. The actual operation of a warm pipeline will of course accelerate thaw within the immediate vicinity of the pipe itself. This additional heat input from the pipeline will have to be controlled by insulation, or in certain cases, by constructing the pipeline above grade on the slope section.

With proper planning, there could be as much as a three-year period from initial tree clearing to actual commencement of pipeline operation, in which case there will be some initial thaw developed across the full width of the right of way. It is likely that in order to minimize the amount of thermal disturbance on sensitive permafrost slopes that the width of the right of way might be limited to 13 m. This will allow for some improvement in the stability of the thawing slope from the side shear effect along the sides of the right of way. It is common to encounter particularly ice-rich conditions in the upper few metres of the permafrost. Therefore this initial thaw before pipeline operation will occur primarily within this more ice rich zone. It may be necessary to consider some form of slope stability enhancement purely for the thawing slope condition described above, independent of what might be required for the additional thaw that will occur because of pipeline operation. The types of mitigation options that could be considered for the sloping right of way itself would include the use of wood chip insulation or other forms of surface insulation. On the more ice rich slopes, it will probably be necessary to insulate the pipe in order to reduce the rate of thaw, and in extreme icy conditions, to construct the pipe above grade.

It will be necessary to conduct detailed drilling investigations to determine the nature of the soils and the actual ice contents, as well as conduct laboratory testing, followed by theoretical stability analyses. The theory involved in stability of thawing permafrost slopes is well established in the literature (McRoberts and Nixon, 1977 and Hanna and McRoberts, 1988). The prime consideration is related to the rate of thaw progression into the permafrost versus the ability of water to drain away from the thaw zone. In relatively fine-grained soils, which tend to contain more ice than coarser soils, the hydraulic conductivity is usually low. Therefore the most significant stability challenge will be encountered with slopes in clay soils that contain excess ice. Slopes in relatively ice rich clays can typically become unstable at angles as low as 7 to 9 degrees, however, it is known that some flow slides have occurred on much flatter slopes - in the order of 3 to 4 degrees - on the Yamal Peninsula in Western Siberia.

If the pipeline is buried on permafrost slopes, a thaw bulb will develop next to the pipe. If significant ice contents extend to the full potential depth of thaw caused by the pipe then it is likely that burial on such a slope will not be feasible. However, since the ice contents often decrease with depth, it may be possible to allow for pipe burial, possibly with the addition of some pipe insulation. In other more critical conditions it may be necessary to construct the pipeline on above-grade supports. However it must be understood that some other measures, such as surface insulation, may still be required to prevent overall instability of the right of way. Since there will be a tendency for some nominal slope movement of the pipeline, the pipeline should be constructed with some longitudinal, expansion loops.

It will be necessary to establish a monitoring program for checking the performance of the more sensitive permafrost slopes. This should include the installation of thermistor strings for monitoring the rate of thaw progression into the slope, as well as some piezometers to observe the development of any excess pore pressures. However, for the most part, the monitoring should consist of the visual observations of the slope surface for evidence of any tension cracks that would indicate any actual slope movement.

4.3.7 Permafrost Foundations

4.3.7.1 <u>Pile Foundations</u>

The selection and design of piles in permafrost depends on many variables such as:

- magnitude of load and allowable settling rate;
- type of soil, ice content, salinity and its temperature;
- the adfreeze strength and shear strength of the soil adjacent to the pile;
- pile type;
- type of pile backfill, if applicable;
- pile installation method;
- availability of construction equipment;
- economics.

The design of piles for the permafrost region is considerably more difficult than in southern, non-permafrost areas because of the presence of ice in the soil structure, its creep behaviors under load, temperature dependence of the creep, seasonal variance of temperatures within the pile depth, presence of saline soil and limitation of construction equipment. These influences interact and have to be taken into consideration to select a pile type and develop a design that is practical and economical.

<u>Load/Time</u>

Frozen soils subjected to load will deform and this deformation is time and temperature dependent. Frozen soils and ice have considerable strength when subjected to short-term loads as produced, for example, by moving trucks or wind gusts. These strengths can exceed by 10 times or greater the strengths under long-term loads. A comparison of short and long-term strengths of typical frozen soil and ice is given in Table 4.1.

Pile foundations are normally designed for a life span of 25 to 100 years. During this time they are subjected to dead loads, live loads, service loads, snow and ice loads and wind loads. These loads can be classified as short-term loads (live, service, snow and ice and wind) and log-term loads (dead). The short-term loads can be designed using a maximum strength of the soil with a suitable factor of safety while the design for long-term loads is based on a strength governed by creep. The allowable long-term strength is obtained from the allowable settlement over the design life of the building. For example this may be 50 mm over 50 years, or 1 mm per year.

Table 4.1 Short-term and long-term strengths of some typical frozen soils and ice (after Voitkovskiy, 1968, as reported by Johnston, 1981)

Soil type	Total water content %	Temperature °C	Uniaxial strength, kg/cm² Short-term Compressive	Short-term Tensile	Long-term Compressive	Long-term Tensile
Medium and fine sand	17-23	−3	60-70	17	6.5	1.8
Silty sand	20-25	−0.3	10-12	5-8	2-3	1.0-1.5
		−5.0	30-40	20-25	6-10	3-5
		−10.0	60-70	40-50	35	11
		−20.0	120-140	50-60	60	21
Clayey silt	20-25	−5.0	23	20	20	9-12
		−10.0	39	30	25	12-15
		−20.0	66	40	40	16-20
	30-35	−3.0	30-35	12-16	3.6	2.5
	35-40	−0.5	8-10	4-6	2	1-2
Clay	25-35	−1.0	15	5	—	1.6
		−5.0	35	13	—	5.0
Polycrystalline ice	100	−3.0	16-20	10-12	0	0
		−10.0	32-40	17-20	0	0

As discussed in Section 3.2, ice or frozen soil subjected to load will creep, which leads to settlement of the pile foundation. The creep phenomenon of ice and frozen soil has three stages which were illustrated on Figure 3.11. The primary stage (I) is characterized by a continuously decreasing slope or creep rate. The secondary or steady-state creep stage (II) is characterized by a constant slope, which is the minimum creep rate reached during the test. Finally, the tertiary stage (III) is characterized by an accelerated creep rate, which normally leads to ultimate failure of the specimen. Pile foundations are designed normally for the steady state creep (II).

The long-term strength of frozen soil has been studied in laboratory shear strength and model testing, and the field pile load tests. The basis for pile design using creep settlement was described by Nixon and McRoberts (1976). Further contributions to the use of adfreeze strength in the design of sand slurry piles was made by Morgenstern *et al.* (1980) and Weaver and Morgenstern (1981). They normalized the pile creep and presented a relationship between average applied shaft stress and normalized pile creep (velocity) for friction piles in ice as shown on Figure 4.3. Design charts are also presented for ice-poor soils by Weaver and Morgenstern (1981).

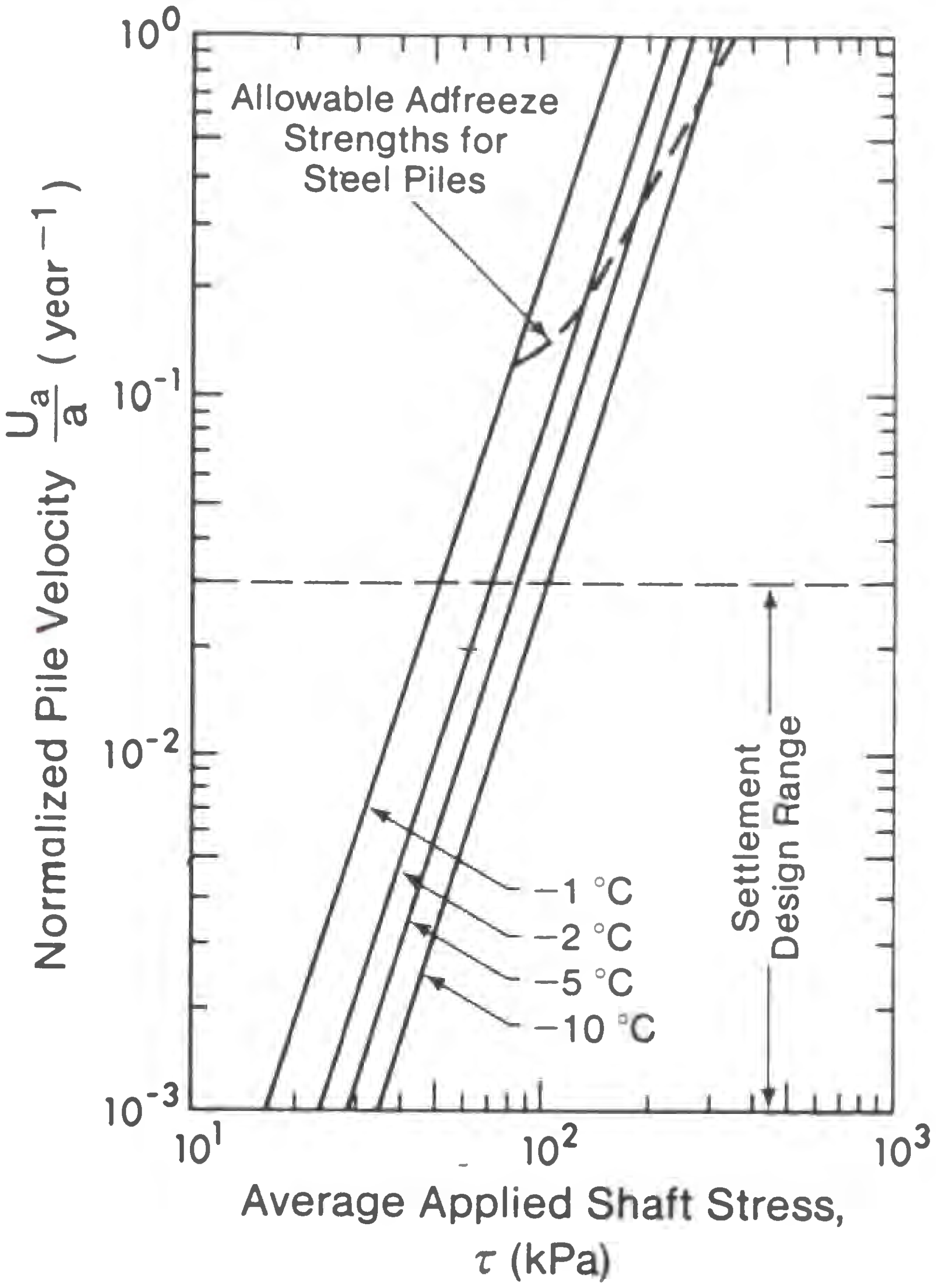


Figure 4.3: Normalized pile velocity versus shaft stress for adfreeze piles in ice (from Weaver and Morgenstern, 1981)

Frost Heave

The annual freeze back of the active layer can subject piles to very large uplift forces, which have been reported by Crory and Reed (1965), and Crory (1963) to be 220 kN and 100 kN on steel pipe piles and wood piles respectively. Field experiments indicate average adfreeze bond strengths of 100 kPa for steel surfaces and 70 kPa for wood and concrete (Johnston, 1981). Peak bond strengths have been measured from 140 to 240 kPa (Penner and Gold, 1971, Penner, 1974) and 40 to 210 kPa by Soviet investigators (Johnston, 1981).

To resist the frost heave forces on piles Heydinger (1987) suggests that pile embedment should be 2 to 3 times greater than the thickness of the soil heave layer.

Thermal Piles

Thermal piles are piles in which natural convection or forced cooling systems have been installed to remove heat from the ground. The thermal devices are normally used with sand slurry piles in warm permafrost. They are used to decrease the time for freezeback, prevent long-term permafrost degradation due to construction damage and decrease the existing ground temperature around piles and therefore increase the pile capacity.

The most common type of thermal piles are the natural convection systems commonly referred to as thermosyphons, thermo tubes, convection cells, heat pipes or air convection. The forced circulation refrigeration systems require external power and mechanical equipment. This type of thermal pile has been used only in special rare cases with very little performance documentation available.

Footings for Elevated Structures

In this section, it must be clearly understood that the structure is sufficiently elevated above the site grade by columns such that there is no direct influence of the heat from the structure on the geothermal regime beneath the structure. The air gap should generally be at least 0.6 m. For square buildings with a plan area greater than about 3,000 m^2, this should be increased to 1 m. Special considerations may be required in regions of extreme snow or snow drifting, since accumulation of snow against the base of the structure may result in building heat being directed downward into the subsurface.

Footings on Permafrost

Footings placed directly on permafrost, sometimes referred to as "Greenland foundations", are quite common in the arctic. The advantage of this foundation is that the footings are installed deep enough so that the bearing surface is below the zone of seasonal heaving and thawing. Generally only relatively light column loads can be supported on such footings. Several aspects must be considered in design, namely:

- soil type, ground temperature, salinity and ice contents
- depth of previous active layer
- depth of post-construction active layer
- insulation requirements
- frost jacking protection on column
- timing of construction

- bearing capacity
- settlement
- maintenance requirements

For sites with ground temperatures warmer than about -2°C, it may not be economical to prevent the permafrost from degrading due to the effects of construction disturbance. Therefore, footings on the permafrost would only be feasible if the ground was thaw stable (i.e. no excess ice).

Sites with ground temperatures colder than -2°C can probably be maintained in the frozen state though insulation may be required for warmer or more ice-rich sites.

Prediction must be made as to the thickness of the active layer following construction disturbance, that is, to establish the "design permafrost table". This can be done by hand calculation (e.g.; Nixon and McRoberts, 1973; Andersland and Anderson, 1978) or by numerical analyses (e.g.; Hwang *et al.* 1972; Nixon and Halliwell 1982). Such predictions must anticipate the long term conditions beneath and adjacent to the building including draining, snow accumulation, warmer surface temperatures along the south wall of the structure, etc. There may also be a potential impact from adjacent site development. The effect of, and the economics of, using insulation to reduce footing depths can also be considered.

Insulation

The use of insulation around footings can have the beneficial effects of "raising" the permafrost table, resulting in possible increased bearing capacity and reduced creep settlement. Alternatively the required depth of footing can be reduced with no corresponding change in bearing or settlement. Figure 4.4 illustrates how the addition of a given amount of insulation can reduce the amount of seasonal warming at the footing level. The required amount of insulation can range from 25 to 75 mm depending on the ground surface temperatures. Bearing capacity and creep settlement potential, being highly temperature dependent in ice-rich soils, are improved by this reduction in warming. Alternatively, the reduced depth of seasonal thaw means the footing can be raised (dashed outline on Figure 4.4), reducing excavation costs and more importantly reducing the potential for groundwater problems in excavations.

Where fill is being placed on a site, the final fill surface is the new reference surface for any seasonal thaw predictions. Where significant fill thicknesses are required or can be accommodated, it may be possible to raise the footing level considerably. Figure 4.5 illustrates that the base of the footing could be placed at 0.6 m below the final grade using 50 mm of insulation instead of almost 2 m depth without insulation. In other words, if 0.6 m of fill were being placed on the site, the footings could be placed on the original grade. For sites with much water in the original active layer, this can be a definite advantage.

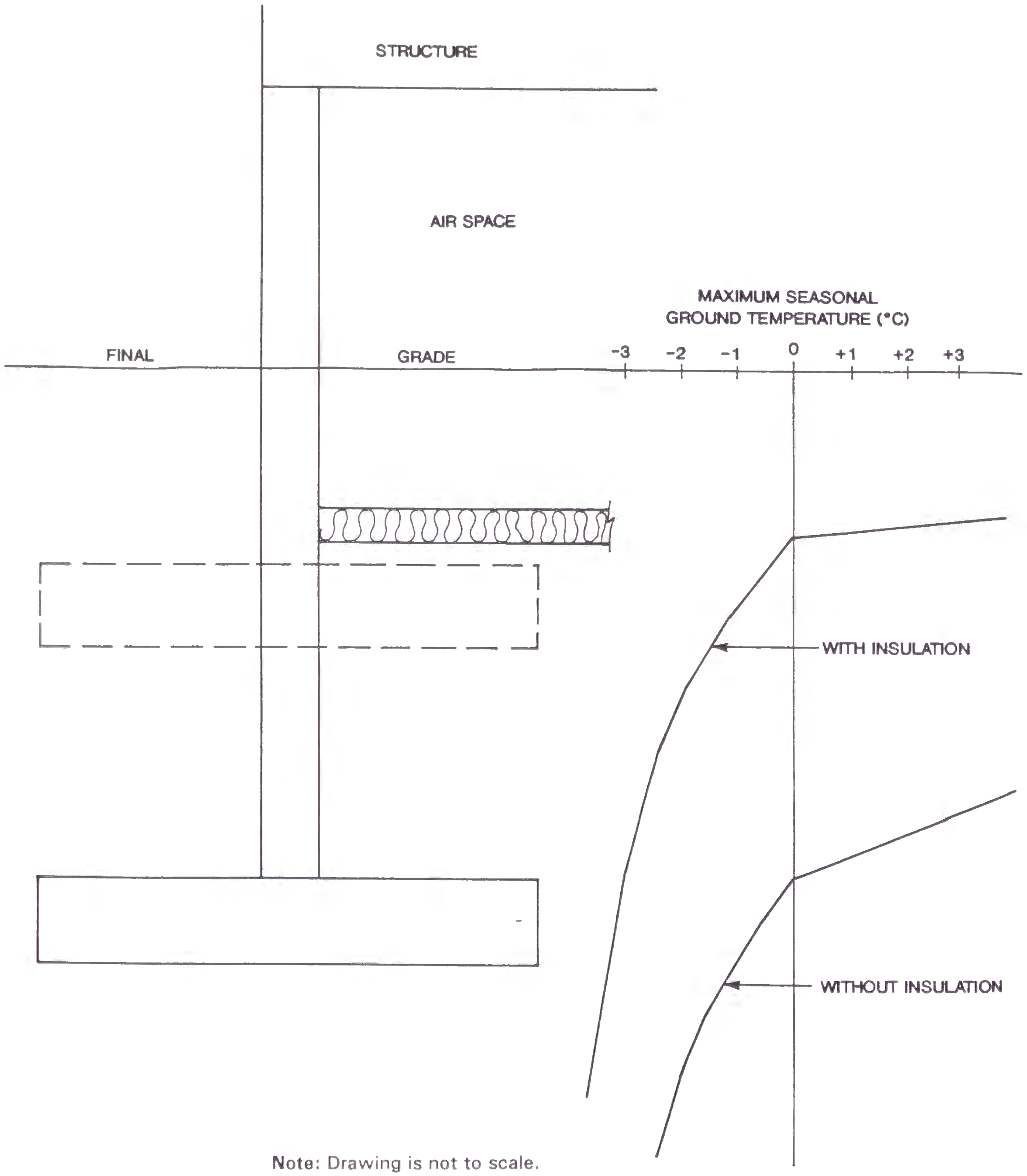


Figure 4.4: Effect of insulation on ground temperature at footing level

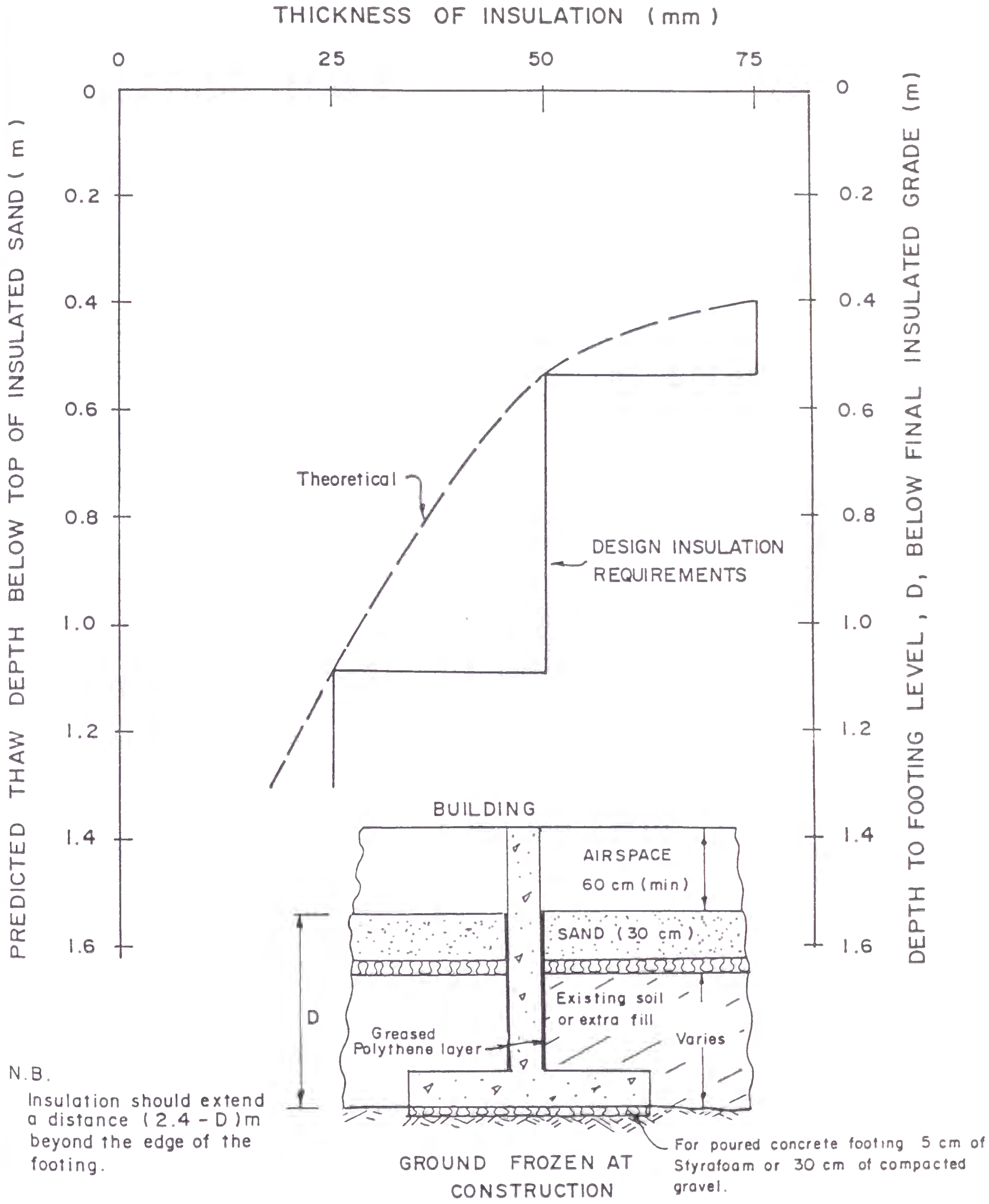


Figure 4.5: Example of required insulation for footing on permafrost

Bearing Capacity and Settlement

The theoretical bearing capacity for footings on permafrost can be calculated as outlined in Andersland and Anderson (1987), or Nixon (1978a). In practice, for most permafrost sites, the actual design bearing capacity will be limited by the allowable settlement rate. Unless the permafrost is adequately confirmed to be sufficiently low in ice content that creep settlement will not occur under the applied load, such creep settlement must be assumed.

Ice-Rich Conditions

The bearing capacity based on a limiting creep rate should consider the warmest temperatures that will be experienced in the zone of influence beneath the footings, which is approximately equal to twice the footing width. This temperature will not be very cold if the footing is at the design permafrost table (Figure 4.4). The temperature will of course be considerably colder at other times of the year. Figure 4.6, which relates the footing settlement to the creep parameters B and n, can be used to determine the acceptable bearing capacity. Since the warmest temperature will only exist for about a month, it is reasonable to consider an allowable settlement rate of 5 to 10 mm per year, depending on how sensitive the structure is to settlement. It is common practice to assume that differential settlement could be about half the total settlement. Based on the allowable bearing pressure obtained from the warmest conditions, calculations can then be made for other times of the year, using the appropriate predicted ground temperature beneath the footing. The actual settlement can then be summed for the year, and if this annual rate of settlement is acceptable, the bearing pressure should be divided by a factor of safety of 1.5. (A relatively low factor of safety is appropriate since designing on the basis of creep settlement is a conservative approach.) Figure 4.6 indicates the significant benefit in having a layer of gravel beneath the footing. For sites with relatively warm ground temperatures and correspondingly higher creep rates, the use of gravel can be cost effective. The use of insulation, as shown in Figures 4.4 and 4.5, can also be advantageous in reducing the creep rate in the warmest time of year.

Ice-Poor Conditions

For sites with friable granular soils, that is the soil is dry enough that it is not bonded, the footing design should be based on conventional unfrozen soil practice.

Where the density of granular soil is greater than around 20 kN/m^3 and well bonded, the bearing capacity can be considerable. At such a high density, creep settlement is not a governing factor. The bearing capacity can be obtained using Figure 4.7. A factor of safety of 3 should be used in this case. This approach should only be used if the high density, bonded conditions are well substantiated. Also, for the higher bearing capacities available under such conditions, qualified construction inspection should be provided.

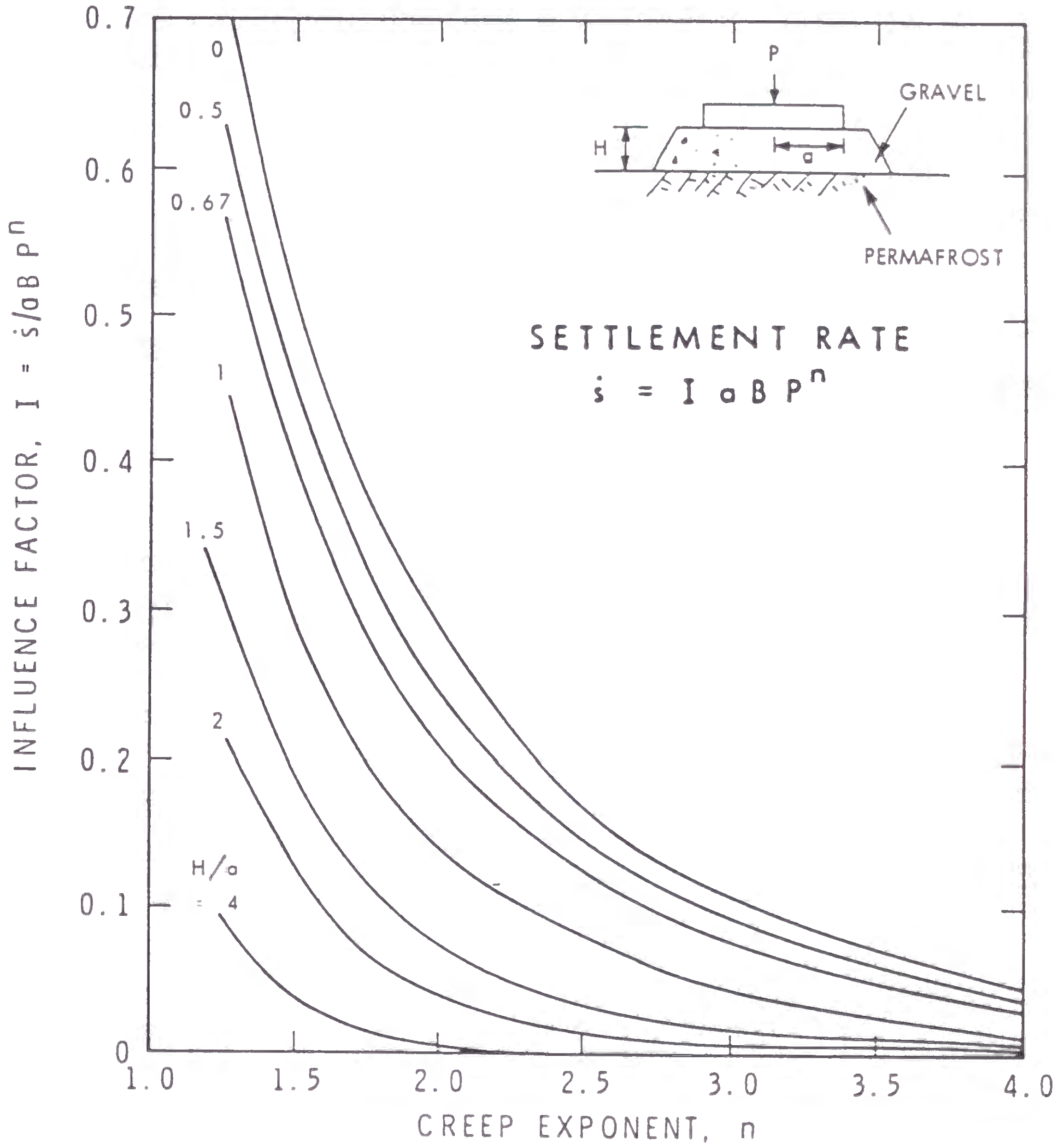


Figure 4.6: Bearing capacity calculation based on limiting creep settlement rate (after Nixon, 1978a)

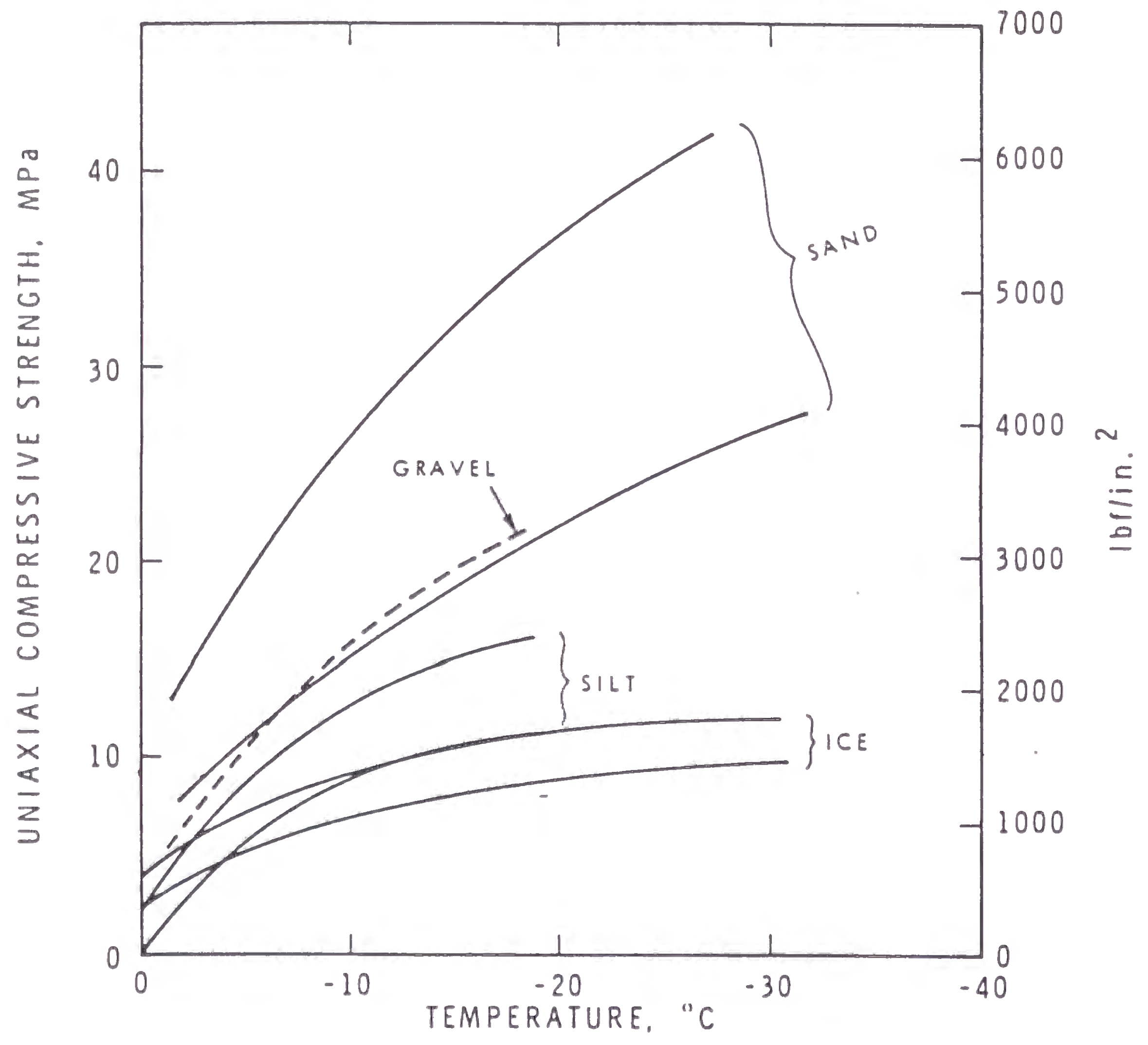


Figure 4.7: Bearing capacity for ice-poor permafrost (from Johnston, 1981; after Mellor, 1972)

Limitations

Whatever the basis for the design, the designer must be aware of the potential for adverse construction conditions and limited construction skills land experience in the smaller arctic communities. For example, footings on disturbed soil or relatively loose fill should be limited to a bearing pressure of about 100 kPa. Qualified inspection should always be provided, however, with some of the noted limitations, it may not be feasible to adequately prepare the bearing surface for the intended design load. It must be realized that for permafrost footings any soft or loose soils will freeze back and then remain frozen under the insulation. It can therefore be permissible to place footings on seemingly poor soil conditions as long as freezeback is allowed to occur prior to load applications.

4.3.7.2 On-Grade Foundations

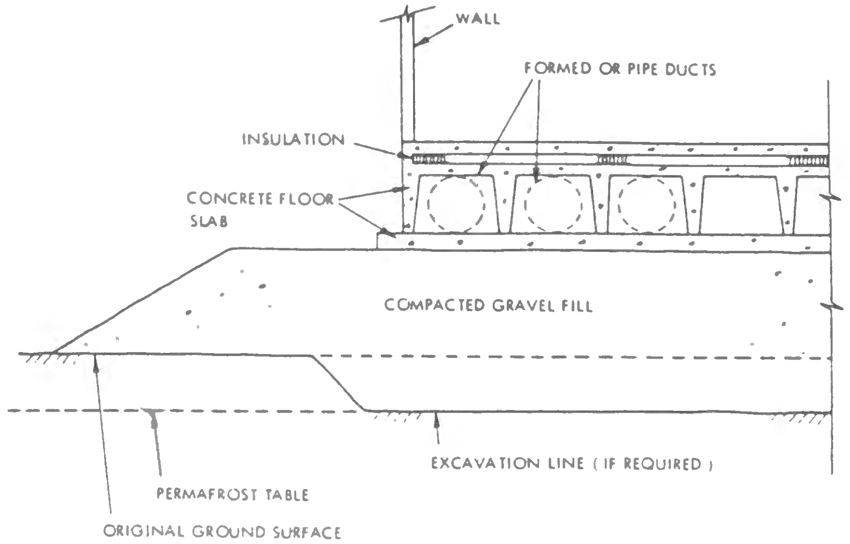
For structures with heavily loaded floors such are warehouses, firehalls, garages, tanks etc., it is common to found the structure on grade, usually with a perimeter beam to support the exterior wall/column loads. Elevated, structural slabs are generally more expensive in such cases. The design is primarily based on the thermal influence of the structure on the underlying permafrost. If the permafrost contains sufficiently low pore ice to be thaw stable, then the design can be similar to southern practice.

For the major of arctic sites, ice-rich conditions prevail or must be conservatively assumed. Building or tank temperatures can vary considerably, from ambient temperatures for fuel tanks to +20°C year round for a firehall for example. It is expected that floor temperatures may be cooler than the specified room temperature but there is no known data to substantiate this. The design approach is based on heat interception. The various methods that have been used to date are illustrated on Figure 4.8. The systems are listed as follows, in order of increasing reliability:

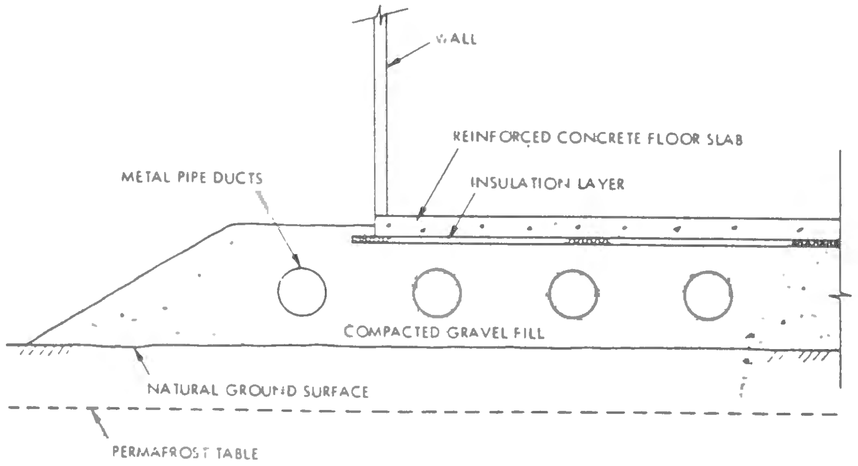
- naturally ventilated and insulated pad;
- forced ventilate and insulated pad;
- insulated pad;
- heat pipes in pad;
- heat pump circulation in pad.

Ventilated Pads

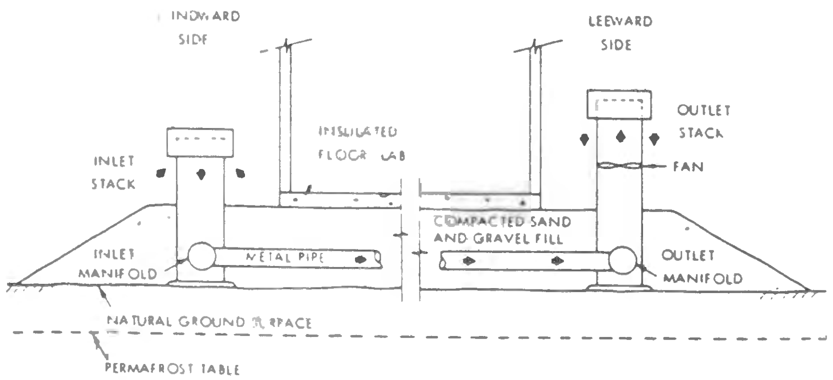
The thermal design must determine the required amount of insulation, spacing of ducts, heat pipes etc. The required inputs are the structure temperature, the floor/fill configuration, the mean ground temperature at depth. An approximate solution for a ventilated pad is available from Nixon (1978b), Figure 4.9. More rigorous analyses should be performed for final design as the above solutions are very general.



Sketch of ducted concrete floor slab on compacted gravel fill foundation.



Sketch of insulated concrete floor slab on duct-ventilated compacted fill foundation.



Sketch of insulated concrete floor slab on forced air, duct-ventilated fill foundation.

Figure 4.8: Natural and forced ventilated heat interception systems (from Johnston, 1981)

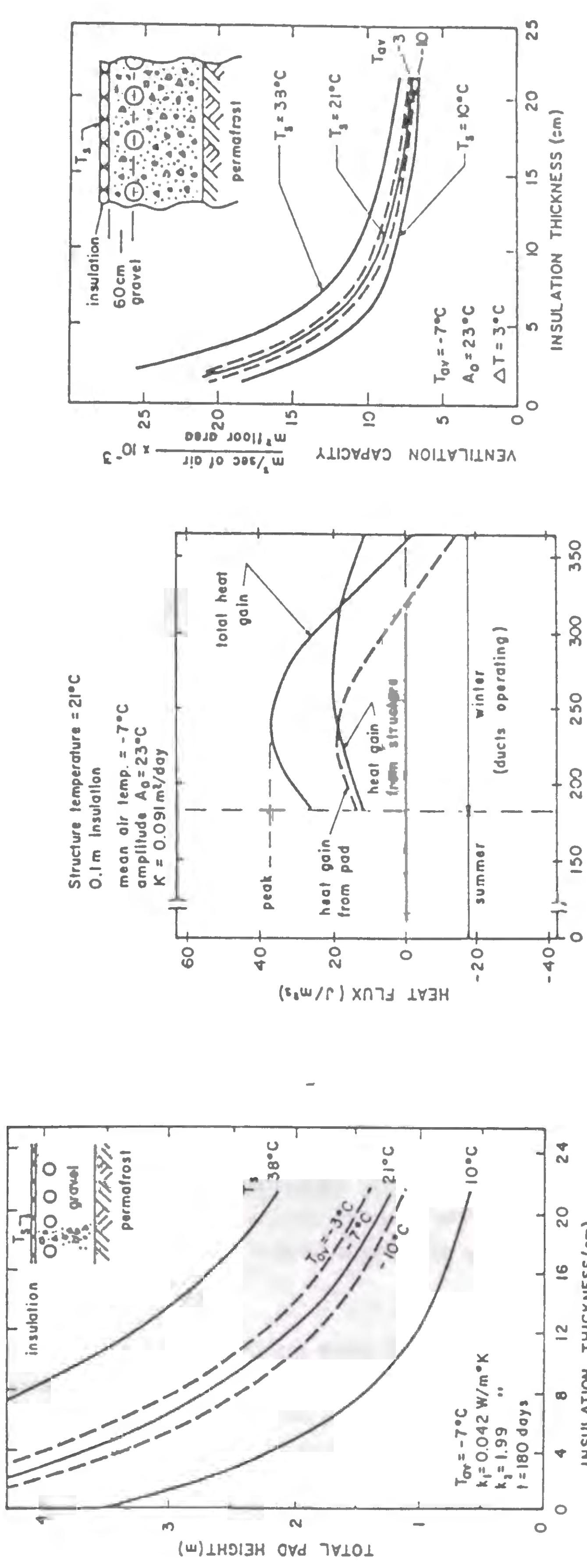


Fig.1 Pad thickness criteria for summer period (ducts closed)

Fig. 2. Sample calculation of heat flux gain to ventilation ducts

Fig.3 Ventilation requirements for cooling ducts

Figure 4.9: General solution for ventilated pad design (after Nixon, 1978b)

In the past, the natural ventilation was used commonly whether with simple open pipes aligned in the direction of the prevailing winds (e.g. Davison and Lo, 1982) or fitted with stacks to create a chimney effect (e.g. Odom, 1983). Problems with snow blockage or insufficient chimney effect led to the use of forced ventilation, however, initially the duct sizes tended to remain large at 450 to 600 mm diameter. Since it is essential to deep the ducts above surrounding terrain for drainage purposes, fill thicknesses to accommodate the ducts were excessive. More recently, a forced ventilated pad has been designed for a garage incorporating 150 mm ducts.

Forced ventilation systems are more reliable than natural ventilation, however, they do require operational checks to ensure the fans are functioning in the winter months. The fans are usually controlled by thermostats linked to both the temperature beneath the flow system and the outside air temperature.

Insulated Pad

An insulation-only alternative can be more reliable, even if it is only feasible over the colder permafrost. A design basis is available for insulated pads, from Nixon (1983) Figure 4.10, and also from Lunardini (1983). It should be noted that this solution is sensitive to the building dimensions and shape. This should be considered when potential expansions are being contemplated. Again, caution should be used with these general solutions and more site-specific thermal analyses should be performed for final designs.

Thermosyphons in Pad

Thermosyphon pipes or their equivalent are used quite commonly for intercepting heat from on-grade structures (e.g. Zarling et al. 1990; Cromin, 1983). A typical configuration is shown on Figure 4.11. These systems are generally efficient and reliable and are well-suited for application where the permafrost is warmer than -3°C, which would be marginal for a ventilated pad and not feasible for an insulation-only design.

The design basis for a thermosyphon pipe system is to some extent similar to that for a ventilated system. Numerical analyses are normally used to determine the optimum insulation requirements and the spacing for a given heat pipe configuration.

4.3.8 Unique Drainage and Erosion Problems

The need for drainage and erosion control measures is dependent upon the velocity of the flowing surface water relative to the allowable non-eroding velocity for that soil. To prevent erosion, the expected velocities of surface water must be prevented from reaching the erosive velocity for the soil.

The expected flows can be determined using the "Rational" method for the design rainfall event. Knowing that the worst case for erosion will be due to concentration of flows into a defined channel, the calculated flows can be set to "Manning's" equation and the channel area determined. Finally, using the continuity equation, the expected velocity can be calculated and compared with the allowable velocity to assess the potential for erosion and therefore the necessity for drainage and erosion control measures.

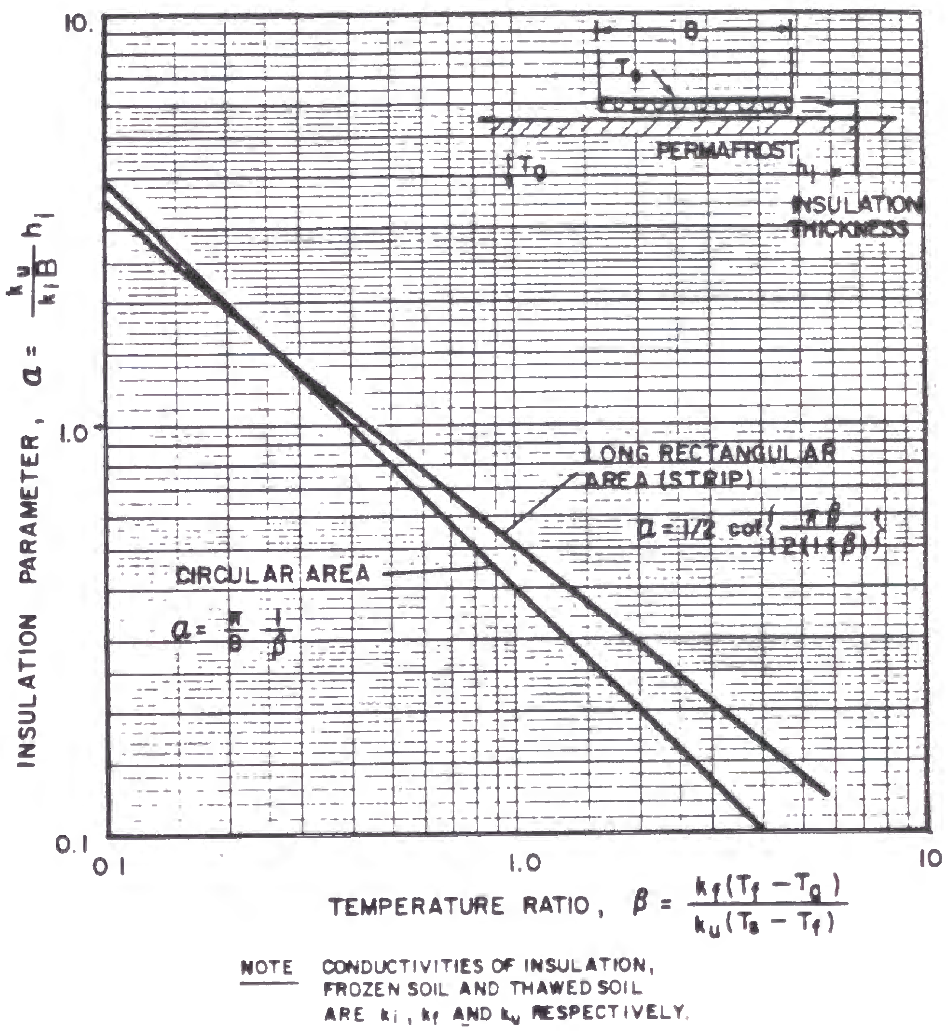


Figure 4.10: General solution for insulated pad design (after Nixon, 1983)

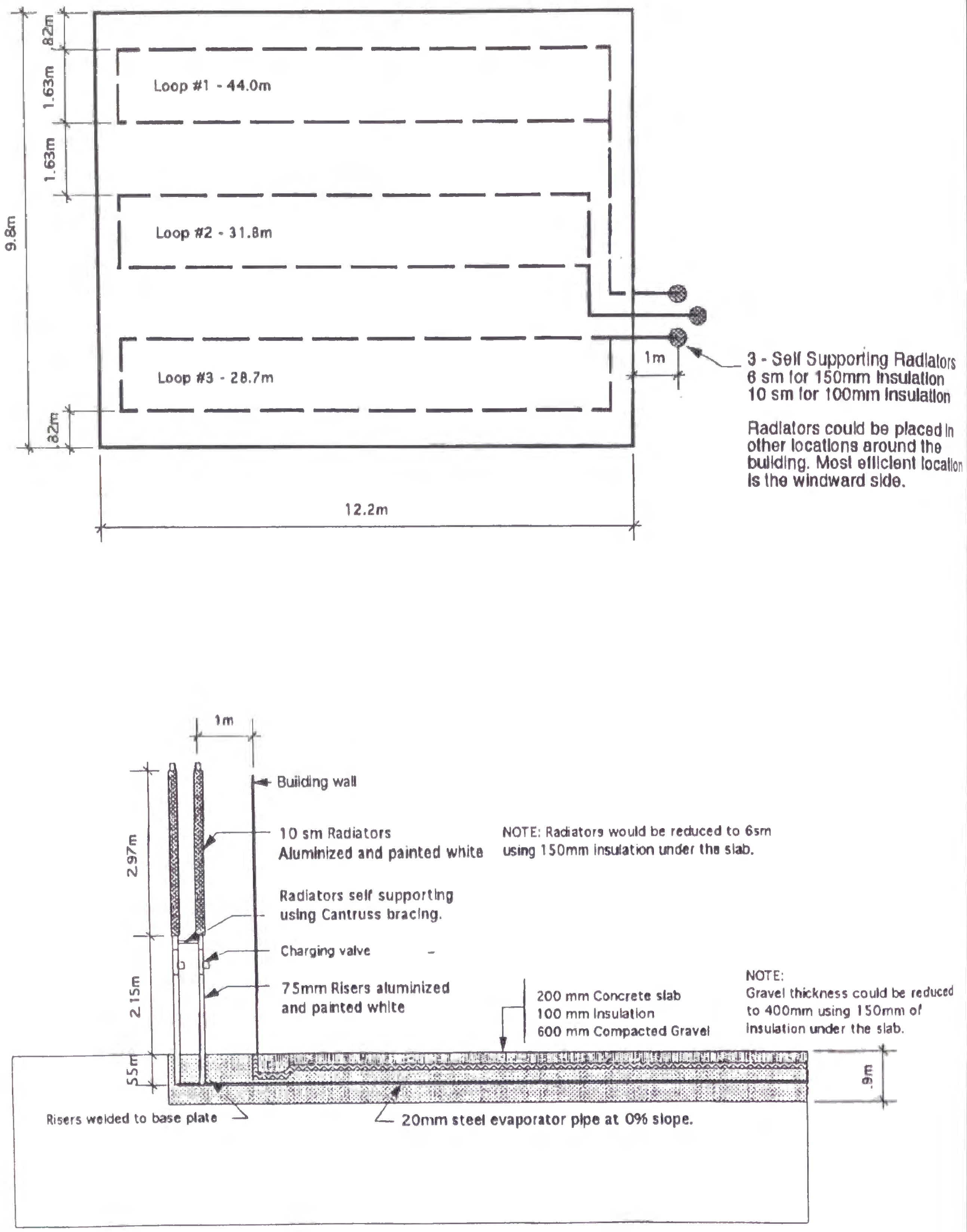


Figure 4.11: Thermosyphon pipe under on-grade structure

(Arctic Foundations, 1997)

Design charts have been developed so that spacings of these structures can be determined. For simplification a single design chart representing average conditions encountered along the pipeline route has been derived, Figure 4.12. The uses and limitations of this design chart have already been presented, however, it can be restated that this design chart can be used to determine the drainage and erosion control requirements for slopes. Therefore, based on the slope gradient, and the approximate allowable velocity of the soil, the spacings of drainage and erosion control measures can be determined.

Typically these measures will consist of diversion berms to improve drainage along the slope and restrict surface water flows to non-erosive velocities, and ditch plugs to restrict the movement of ground water through the pipeline ditch by forcing the groundwater within the ditch to the surface where it can be diverted off the right of way. Run off diversions will be used to restrict flows across slopes to non erosive velocities, particularly on cut-slope and insulated slope designs.

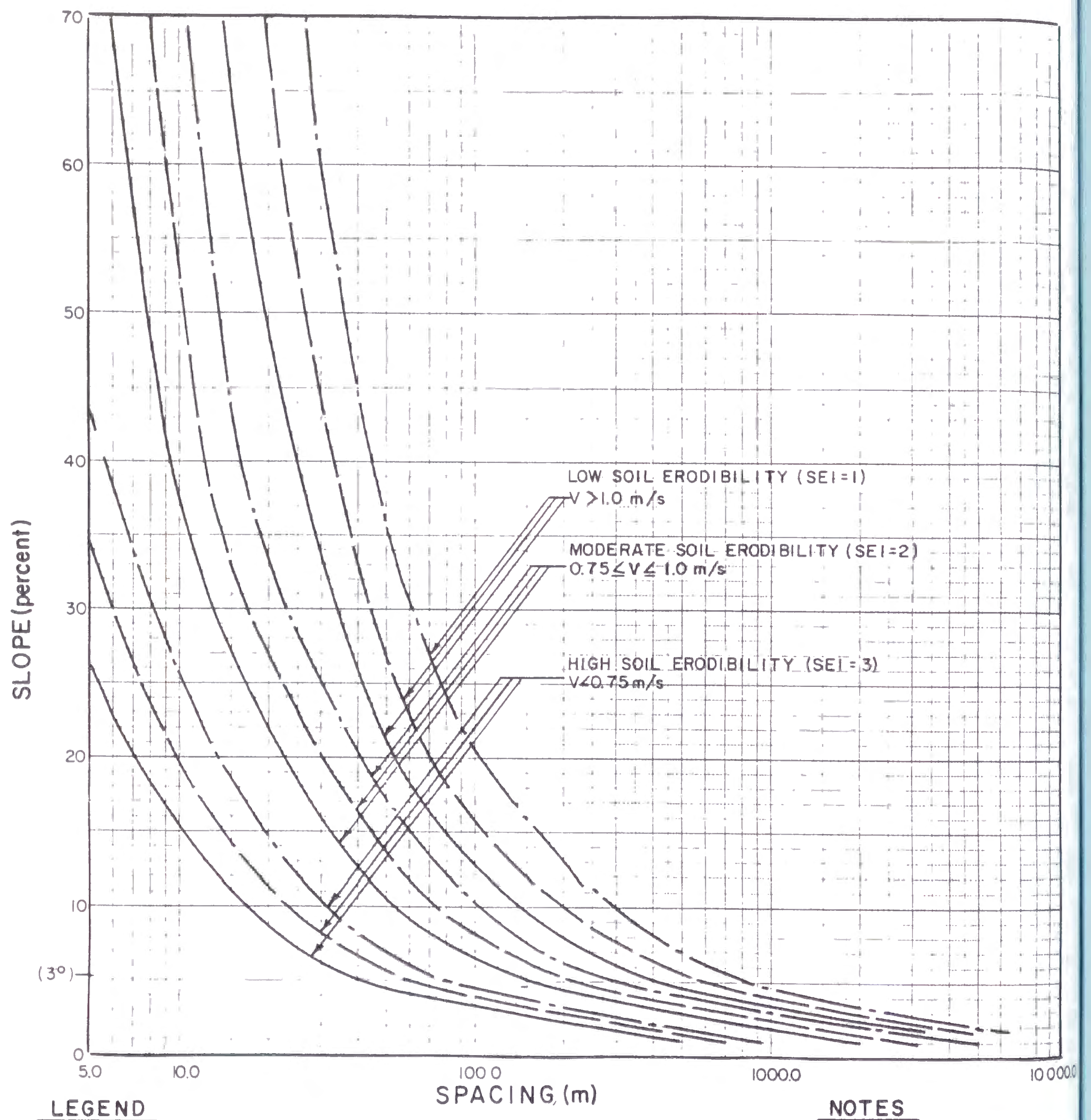
The requirements for drainage and erosion control can be predetermined, however, the exact placement of these requirements is to be performed by the field engineer to account for post-construction geometry and the intent of the drainage and erosion control mitigation.

The construction of any linear facility has the potential to alter the local drainage pattern all along the route. The clearing of trees and disturbance to the surface vegetation also alters the nature of the "run-off" of rain water or snow melt from the right of way. In areas of extreme disturbance, where the mineral soil is exposed, the soil surface may be highly erodible. The backfill over the pipe will mostly comprise of the native soil that was excavated from the trench. For winter-construction spreads, the backfill will have been placed in frozen clumps and will be highly porous until it has thawed and settled, during the first summer following placement. This backfill mound will be extremely erodible during the first spring after construction. The right of way should be seeded immediately following construction, however, until vegetation becomes properly re-established, the surface will be more erodible. Typical means of controlling erosion on the ditchline and along the right of way include:

- ditch plugs
- mound breaks
- diversion berms

There is another form of erosion that can occur in permafrost regions, namely thermal erosion. This occurs as relatively warm summer water flow causes thawing and the thawed soils are usually highly erodible. The combined thermal and physical erosion can lead to very extensive erosion in silty, sandy soils, even on relatively low gradient slopes.

The right of way will also intercept numerous cross-drainages, in addition to the more obvious creeks and rivers. Many of these cross-drainages may not flow all year or every year. Almost certainly most will not be evident during winter construction. It will be important to anticipate erosion at numerous unidentified cross-drainages, which will require maintenance in the first few years following construction.



LEGEND

- · - ZONE 3 Russia
- — — ZONE 2 Mongolia
- —— ZONE 1 China

V = ALLOWABLE VELOCITY
n = MANNING'S ROUGHNESS COEFFICIENT
c = RUNOFF COEFFICIENT

NOTES

1. CHART ASSUMES AVERAGE CONDITIONS
2. BASED ON 1:50 YEAR RETURN EVENT
3. SPACING IS FOR PARALLEL FLOW.
4. FOR CROSS FLOW, MULTIPLY SPACING BY 25 TO OBTAIN MAXIMUM DRAINAGE AREA FOR MOUND BREAKS.
5. ASSUMES n = 0.03 AND c = 0.4 .

Figure 4.12: Typical diversion berm and mound break spacing design chart

5.0 FEASIBILITY STUDY ON PROPOSED NATURAL GAS PIPELINE FROM EAST SIBERIA.

The main contribution of this thesis is to apply the approaches that the author has learned, as summarized in the foregoing sections, specifically to studying the feasibility of designing, constructing and operating a natural gas pipeline from the Irkutsk region of Eastern Siberia to the Chinese Yellow Sea coast. While the study considered the entire pipeline system, much of the emphasis is on the permafrost design, construction and operations issues.

5.1 ROUTE SELECTION

The Leno - Viluyskaya oil and gas province and the Leno-Angarskaya gas province are located mostly in the discontinuous permafrost area, as shown on Figure 5.1. For the purposes of this study, the city of Ust'-Kut is proposed as a focal gathering point for the natural gas. The main transmission pipeline will therefore start at Ust'-Kut.

The main philosophy for the route selection for major transmission pipelines is to obtain the shortest possible route to the market location. In this case the market being considered is the Beijing area and offshore export to South Korea and Japan. The coastal terminal will be at Tianjin, just southeast of Beijing, as shown on Figure 5.2.

In the initial pipeline section from Ust'-Kut, there are major geographical constraints to the selection of a pipeline route. These constraints are Lake Baikal, the world's largest fresh water lake and the Khamar-Daban mountain range. Lake Baikal has been designated a major ecological reserve, in an attempt to preserve the quality of the water. The Khamar-Daban mountain range rises to as much as 2,500 m in elevation, or about 2,000 m above the lake level. There is no major pass through this mountain range near the proposed pipeline alignment.

Clearly it is difficult for the pipeline route to follow the straightest possible alignment between Ust'-Kut and Ulaan Baatar. The proposed route must make a detour around the southwest end of Lake Baikal and then head east to northeast around the Khamar-Daban Mountain Range to Ulan Ude. Then the pipeline follows the main transportation corridor through various valleys to Ulaan Baatar.

Apart from these major detours, it is a major benefit to chose an alignment close to existing transportation corridors, to facilitate the transportation of pipe materials and construction equipment. Other factors that will influence the pipeline alignment to some extent will be the location of major market nodes along the alignment. However, in most cases it is more economical to construct smaller diameter laterals from the main line to certain market nodes depending on the size of the market and the relative location.

The proposed pipeline alignment is shown on a set of alignment sheets (reduced to 50% of the original drawing size) and are included in Appendix B. The alignment shown on these maps has not been checked in the field, therefore, the alignment is to be considered preliminary. The line drawn on the alignment sheets is an indication of the pipeline corridor that is being considered for this study.

Figure 5.1: Location Of Gas Provinces

Scale 1: 12,000 000 (Approx.)

Figure 5.2: Pipeline Route Along Railway & Roadway

It is expected that in some of the more restricted, narrow valleys or steep side-slope areas, the selection of an optimum alignment may be a challenge, such as at Kilometre Post (KP) 650, KP 920 and KP 2650.

The selected routing as shown on the alignment sheets is a total of 2 875 km long from Ust'-Kut to Tianjin. For this pipeline alignment, 1 152 km is situated within Russia; 958 km is within Mongolia and 765 km in China.

Some significant refinements to this alignment should be considered. The most notable of these will be an attempt to make a short-cut across the Khamar-Daban Mountains. An alignment, that has apparently been considered by others, would run from Babushkin, at approximately KP 790, to Selenduna at KP 1090. This would amount to a shortening of almost 200 km in the overall length of the pipeline and must be given serious consideration. There is a road that passes through the mountain range to a height of at least 1 500 m, asl, or about 1050 m above the lake level.

Although it is known that Lake Baikal is a significant environmental protection area it should be noted that the pipeline could be shortened by almost another 200 km if a lake crossing was to be considered. A crossing from the north shore at Bol'shoye Goloustnoye to Babushkin on the south shore would in fact provide almost the optimum alignment for the pipeline between Ust'-Kut and Ulaan Baatar. Besides the environmental concerns, the crossing of Lake Baikal would present major design and construction challenges, on account of the great depth and steep submarine slopes.

In future studies, more attention will also have to be provided to detailed route selection near populated areas. In general, the pipeline should be routed well outside any presently populated or potential expansion areas around towns and cities. Consideration should also be given to the requirements for gas supply to the various populated zones along the alignment, in addition to the major centres considered in this study - Irkutsk, Ulaan Baatar and Beijing.

5.2 PERMAFROST AND TERRAIN CONDITIONS ALONG ROUTE

5.2.1 Permafrost Distribution and Conditions

Based on the information reviewed, approximately half the pipeline route is situated within the discontinuous permafrost region and is therefore underlain by intermittent permafrost. As expected, the permafrost is more widespread near the north end. The most southerly occurrence of permafrost is shown on Figure 5.2, just south of Ulaan Baatar. The range of permafrost conditions that require design consideration are summarized as follows.

The pipeline route from Ust'-Kut to Kultuk (KP 620) crosses the Angaro-Lenskiy Geocryological Region. Typical locations for permafrost are swampy bottoms of valleys and the lower portions of wooded slopes with northern exposure. The percent of permafrost distribution increases where the elevation is higher. For example, permafrost occupies 10 to 20% of the area for elevations from 500 to 600 m and consists of 30 - 35% of the area, where the elevation is more than 800 m. The mean annual temperature of permafrost varies from -0.1 to -2 °C. Atmospheric temperature inversion is typical at elevations of 500 to 800 m, therefore the soil temperature at elevation 500 m is lower than at elevation 800 m.

Normal temperature distribution is observed at the elevations more than 800 m; the soil temperature at elevations of 1 000 to 1 100 m (maximum for this region) is 0.5 to 0.8 °C colder than at elevation of 800 m.

Silty and clayey soils in the river valleys have moderate to high ice content. The thickness of the ice layers can reach 3 to 5 cm and the moisture content varies from 30% to 70%. Frozen soils at the higher ground and slopes have low ice content, especially at elevations of more than 800 m. As a rule in these areas there are no visible ice lenses and layers. Moisture content varies from 25% to 35%.

From Kultuk to Ulan-Ude (KP 620 to 940) the route crosses the Baikal Geocryological Region. This territory borders on Lake Baikal and so the geocryological conditions are moderated. The isolated islands of permafrost are encountered on north facing slopes and at the swampy, peaty areas of river valleys. The percent of permafrost distribution does not exceed 10 - 15%. The mean annual temperature of the frozen soil varies from -0.1 °C to -0.5 °C. As one moves away from the lake, the percent of permafrost occurrence increases to 20 - 25% in the Khamar - Daban ridge near Ulan-Ude. The frozen soil temperature equals -0.7°C to -1 °C.

The ice content in alluvial soils in the river valleys varies from moderate to high; moisture contents are as high as 35 - 70%. The ice content in the bedrock and weathered rock is generally low and the moisture content does not exceed 25 -30%.

The pipeline route from Ulan-Ude (KP 940) to KP 1300 (south of Darhan, Mongolia) crosses the Selenga - Orkhon Geocryological Region. This region is characterized by sporadic distribution of permafrost. The rare islands of permafrost are found in moist depressions, which have been formed in silty and clayey deposits, and in river valleys. Permafrost occupies less than 10% of the terrain and has temperatures to -1 °C.

The maximum ice content is noted in alluvial deposits to a depth of 10 m. Ice layers can have a thickness of 0.5 - 2 cm and the total moisture content of the frozen soil reaches 80 - 100%. The ice content of the frozen soils in moist depressions, which are typical for the Mongolian portion of the region, is lower. Frozen soil generally has no visible ice layers and lenses and the total moisture content is not more than 15 - 25%. The ice content of bedrock depends upon the degree of fracturing, porosity and stratification. Ice in bedrock can be found to the depth of 15 m contained in fractures, pores and joints between different lithological strata. Total moisture content equals 15 - 25%.

From KP 1300 to the southern limit of permafrost (KP 1650), the proposed route is located in the Hentiyn Geocryological Region. The Hentiyn Nuruu mountains are characterized by a discontinuous distribution of permafrost (20 - 40%). From KP 1530 (Ulaan Baatar) to the southern limit, permafrost can be found only near springs, where moisture content of soil is higher. Total area with permafrost less than 5%, and the annual mean temperature of the frozen soils -0.1°C to -0.5 °C. The frozen soil contains a high volume of ice with total moisture contents exceeding 50%.

5.2.2 Specific Permafrost Conditions - Angaro-Lenskiy Region

Details have been obtained for existing boreholes in the geocryological region at the north end of the proposed pipeline system. The data characterizes permafrost properties in the Ust-Ilimsk area, located in the Angara valley 200 km to the northwest of Ust-Kut (Figure 5.3). The logs for 12 boreholes are presented in Appendix C. The geocryological conditions in the Ust-Ilimsk area are very similar to the conditions of the Ust-Kut area.

The sites are located on the middle to high alluvial terraces of the Angara river, some 50 to 170 m above the river level. The surficial deposits are diluvial and alluvial sand, silt, silty clay and clay. The thickness of the diluvium is up to 5 m. The underlying alluvium has thicknesses ranging from 3.5 to 6.0 m. These strata are underlain by a weathered to unweathered sandstone bedrock.

The thickness of the active layer is 1.5 to 2.2 m in clay and silty clay (Boreholes 29, 41127, 41128, 41976, 41977). The sand, silt and the weathered sandstone have active layer thicknesses to as much as 3.2 to 3.9 m (Boreholes 7, 71231, 71232).

The maximum moisture content (35 -55%) is reported in the clays and silty clays. Ice in these soils is generally in the form of layers with thicknesses up to 1 cm (Boreholes 25, 26, and 29). The silts, sands and weathered sandstone have no visible ice and the moisture content is 10 to 22% (Boreholes 71231, 71232). Near the bottom of the alluvial deposits (at depths of 6 to 8 m), moisture contents typically do not exceed 10 to 15%. The sandstone (bedrock) has moisture contents of 10 to 15%. Figure 5.4 shows the variation in moisture contents with depth in the unconsolidated soils overlying the sandstone. Any moisture contents greater than about 30%, indicate excess ice in the finer grained permafrost, which will cause settlement on thawing. In the coarser, silts and sands, moisture contents greater than about 22% indicate excess ice.

The mean annual permafrost temperature is in the range of -0 to -0.8°C, indicating that the permafrost will be sensitive to surface disturbance from pipeline construction and operation.

Figure 5.3: Borehole locations in the Angaro-Lensky geocryological region

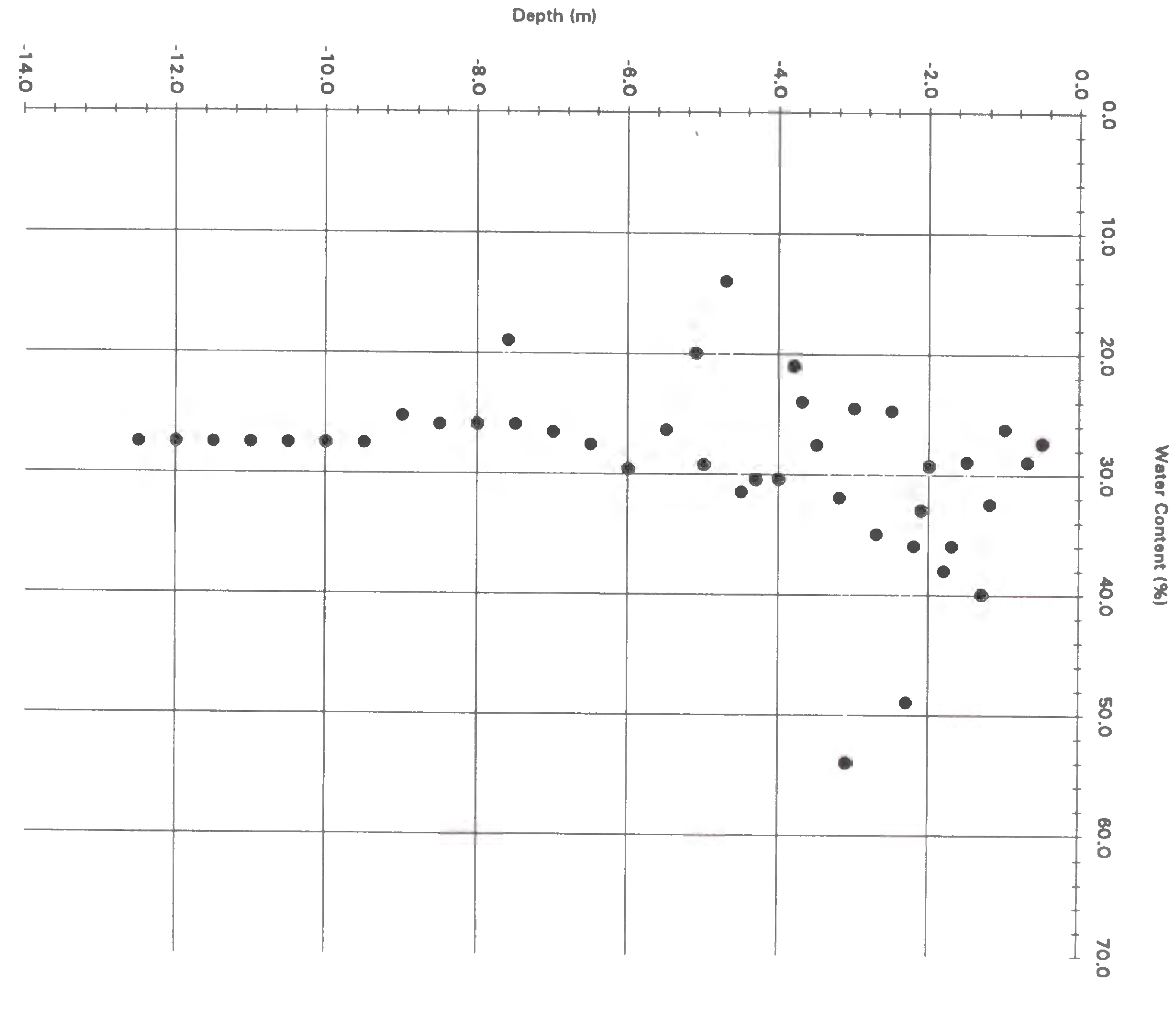


Figure 5.4: Water content versus depth

5.2.3 Geotechnical Design Parameters

Based on this specific soils data for the region, the following representative soil parameters are selected for the purposes of the design of the pipeline and the related facilities:

Basic Soil Strata and Moisture Cont.	0.0 to 4.0 m	silty clay (40% m/c)
	4.0 to 9.0 m	silty sand (26% m/c)
	9.0 and deeper	weathered sandstone (12% m/c)
Thaw Rate, α	3.4×10^{-2} cm/sec^{½}, for a surface with little disturbance	
	6.1×10^{-2} cm/sec^{½}, for a surface with much disturbance	
Coeff. of Consolidation, c_v	1×10^{-3} cm^2/sec, for typical silty clay soil	
Thaw Consol. Ratio, R	0.34, for a surface with little disturbance	
	0.61, for a surface with much disturbance	
Thaw Settlement	0.15 average thaw strain	
Apparent Cohesion, c	3.5 kPa	
Internal Friction, ϕ	24.5 degrees	

5.2.4 Bedrock

Soil and rock information along the alignment are shown on the lower portion of the alignment sheets, Appendix B. This information has been obtained from various types of geological and geomorphological maps at scales as small as 1:5 000 000. The main purpose of this information is to identify areas where rock might be encountered during ditching for the pipeline. An attempt has been made to differentiate between "soft" rock (usually sedimentary) which is expected to be excavatable, with perhaps some assistance of rippers. In other zones, a "hard" rock category (metamorphic/igneous) has been identified where it is expected that "blast-assist" might be required in the ditching operation.

5.2.5 Seismicity

It is understood that much of this pipeline alignment is situated within relatively active seismic zones. There are similar seismic zoning systems within each of the three countries. The actual seismic zones are shown on the upper portion of the elevation profile on the alignment sheets, Appendix B.

The proposed route traverses zones with intensity of seismic influence from Zone 6 to 9, with the maximum intensity of seismic influence being encountered near Lake Baikal.

This seismic zoning relates to soils with average seismic properties, defined as:

- weathered bedrock, frozen and unfrozen;
- gravelly, pebbley and rubbley soils with filler less than 30%;
- gravelly, coarse, medium sands, dense and medium dense, dry and moist;
- fine and dusty sands, dense and medium, dry;
- clayey soils with consistency ≤ 0.5 and porosity coefficient ≤ 0.9 (for clay and silty clay) and porosity coefficient < 0.7 (for sandy silt);

- plastic frozen, dry frozen and hard frozen (with temperature above -2°C) soils, if used as a frozen foundation support.

For soils with more favourable physical properties (e.g., massive rock), the Zone designation may be reduced by one. For soils with less favourable physical properties (e.g., soft, loose soils) the Zone designation should be increased by one. According to the Russian SNIP II - 7- 81, the seismic influence on buildings and facilities should be taken into account, for Zone 6 or greater. Construction in terrain with a seismic zone higher than 9, must be approved by the Ministry of Construction of the Russian Federation.

Some geological maps referred to in this study show fault zones in the vicinity of the pipeline alignment, however, it is not known whether these are considered currently active. This will require discussion with local geologists and field reconnaissance.

5.3 PIPELINE DESIGN FOR PERMAFROST CONDITIONS

5.3.1 Design Basis

It is proposed that the pipeline and facility design be based in Western codes and standards. The design basis specifies the major data and criteria for the design of the pipeline. This includes the flow forecast, as well as the specific requirements or constraints on design, construction, or operation. Design requirements include items such as a specified MAOP (maximum allowable operating pressure), specified points that must either be included or avoided on the route, and specified types or grades of pipe or other procurement constraints.

The following are the main items for the design basis:

<u>Pipe Diameter:</u>	56" or 1 422.4 mm	
<u>Maximum Allowable Operating Pressure (MAOP)</u>		9 930 kPag
<u>Gas Receipts:</u>	Ust'-Kut	33.0×10^9 m^3/year
<u>Gas Deliveries:</u>		
	Irkutsk	0.5×10^9 m^3/year
	Ulaan Baatar	0.5×10^9 m^3/year
	Beijing	10.0×10^9 m^3/year
	Tianjin (export)	<u>22.0</u> $\times 10^9$ m^3/year
	TOTAL	33.0×10^9m^3/year

5.3.2 Flow Forecast

For the proposed pipeline, the long term flow forecast is based on the following information. The gas will be collected and processed at Ust'-Kut. The various gas fields are sufficient to ensure gas receipts by the pipeline company of 33.0×10^9 m^3/year. The gas will be pressurized and cooled to 10°C for input to the transmission pipeline.

There are four main delivery points on the pipeline - the first two, at Irkutsk and Ulaan Baatar, are for small quantities of 0.5 x 10^9 m^3/year; the main deliveries being 10.0 x 10^9 m^3/year to Beijing and 22.0 x 10^9 m^3/year for export to South Korea and Japan.

The long term forecast has to account for the annual, seasonal, and possibly even diurnal patterns in flow. The widely used method of doing this is in terms of a throughput factor. The expected throughput factor, sometimes called the load factor, is an important complement to the basic flow forecast in terms of expected annual volumes. The term "throughput" factor is preferable to "load" factor since reductions of pipeline flow can arise from many sources (though they are most often attributable to load characteristics). The throughput factor is a measure of the variability of flow. It is defined as the ratio of the average flow to the maximum flow, generally on an annual basis. It is generally less than 1.00 and varies from one pipeline system to another. The significance of the throughput factor is that generally the sizing of pipeline components is determined by the maximum flow but the revenue is determined by the average flow.

A throughput factor of less than 1.00 arises from three principal sources:

1. Fluctuation of gas demand
2. Interruption or reduction of gas supply
3. Interruption or reduction of pipeline capacity

Most often, the first source, fluctuation of demand for gas from users connected to the pipeline system, is the dominant one in determining the pipeline throughput factor. Fluctuations in demand depend on the end use of the gas. For all types of demand very short-term fluctuations on a daily or shorter cycle can generally be met from line pack and do not affect overall pipeline throughput. Longer-term fluctuations reduce throughput factor (unless gas storage, such as salt caverns, is available near the demand sites). This occurs because the pipeline must be sized for the peak demand flow but periods of reduced demand result in reduced pipeline flow and throughput factor.

Gas demand is commonly subdivided into three categories of users: industrial, commercial, and domestic. Gas demand for industrial loads (such as electrical power generation, chemical plants, and so forth) is usually nearly steady. However, there may be some variation in the annual consumption pattern. For example, in some climates, the electricity demand for air conditioning will show a seasonal pattern. Commercial and domestic loads (other than space heating) are somewhat more variable than industrial loads but still relatively steady. The chief source of fluctuation in pipeline demand comes from space heating loads that closely follow the seasonal weather pattern. The consumption of gas for space heating is common to all three categories of gas users, however, the large numbers of households, small buildings (i.e., high exposed surface to volume ratios), and the lower efficiency of small domestic gas heaters, usually cause the domestic portion of space-heating load to be the largest.

The second source for reducing the throughput factor, interruption or reduction of gas supply, is seldom a large factor in determining pipeline throughput factor. There are many reasons for this, including:

1. The design capacity of the gas supply system will be set equal to or greater than the pipeline capacity and its connected gas demand. The gas supply and pipeline design capacities will be matched.

2. For large pipeline systems, the gas will probably originate from multiple gas plants, gas fields, and gathering systems. A problem causing reduction of supply is likely to be confined to only part of the originating system.

3. Critical facilities in the gas supply system usually have spare capacity or redundancy.

4. Compensatory strategies permit short-term adaptation to increase supply from other sources if supply from some sources is curtailed.

Reductions (or interruptions) of gas supply can be categorized as planned and unplanned. Planned activities can be distributed both in time and space (e.g., throughout the year at different locations) in order to spread their effects more uniformly or to minimize their impact. For example, planned maintenance at the originating gas plants could be scheduled to coincide with periods of reduced gas demand. For both planned and unplanned events, the availability of spare facilities may prevent a reduction in gas supply to the main pipeline.

The third potential source of throughput reductions comes from the passive portions of the pipeline system, such as the pipe itself. These are seldom the cause of throughput reduction in a well designed and maintained system. Special precautions may be taken in more vulnerable sections, such as river crossings, where the redundancy strategy may be used (e.g., dual crossings). In pipelines traversing permafrost susceptible to thaw settlement, preserving and ensuring the integrity of the pipeline will imply more monitoring and maintenance.

It is the rotating equipment at stations that usually requires the broadest application of the sparing and redundancy approach in order to preserve pipeline throughput capacity. Typically a station will comprise multiple compressor-driver sets in a, 2 x 100% or 3 x 50%, capacity configuration. Other strategies could include larger units and/or closer station spacing.

If all these measures are applied, pipeline availability approaching 100% can be attained, implying that pipeline capacity need not be the determinant of throughput factor.

For the proposed gas pipeline system, which is driven by demand, the recommended design throughput factor is 90% on an annual basis, following a sine wave pattern with a trough of 80% in the summer and a peak of 100% in the winter. The throughput factor exerts a strong influence on design, especially regarding items such as:

1. Station overall power and spacing
2. Compressor unit driver selection
3. Chilling and cooling system scale and configuration.

5.3.3 Hydraulic Analysis

The hydraulic analysis required for such a pipeline is usually performed with a computer program that uses a finite difference method to solve the primary hydraulic equations in a piecewise step method along the route:

1. Continuity equation

2. Momentum equation

3. Energy equation

together with the defining, Equation of State for fluid properties:

There are a variety of supplementary equations that are also used, including those for gas transport properties (such as viscosity), compression at stations, and cooling or heating. Individual compressor units at stations were modelled using a composite efficiency value of 80% assigned for each station.

The momentum equation accounts for pressure drop along the pipeline arising from gas law variations, elevation changes, and principally from frictional losses. The form of the equation used in the computer program is the AGA equation, which is a rational equation rather than an empirical one such as Panhandle. This has the advantage that the equation is dimensionally correct in terms of powers of various terms (which facilitates dimensional sensitivity studies). The form of the complementary friction factor (expressed as a Fanning friction factor) follows the Colebrook-White formulation for turbulent flow with the friction factor a function of Reynolds number, pipe internal roughness, and diameter.

The energy equation accounts for temperature changes along the pipeline. These arise from two main sources:

1. Heat flux to the ground caused by the difference between the flowing gas temperature and the ground temperature

2. Cooling of the gas due to the Joule-Thomson effect as pressure along the pipeline drops from the discharge of each station to the suction of the next.

The heat flux to the ground is calculated using a two-dimensional cross section of the pipeline with homogeneous ground and ambient air temperatures. Moreover the soil is characterized by a single conductivity value at each cross-section along the pipeline. This simplified method can give excellent results compared to those encountered in the real world as long as appropriate values for the temperature and conductivity parameters are chosen. For this pipeline, the influence of these values is not especially high. Heat flux to the ground accounts for only a small fraction of the total heat energy added to the gas by compression; most heat must be removed by chilling or cooling.

Station power is calculated assuming an isentropic compression using calculated gas properties of entropy and enthalpy. The effect of efficiency is then accounted for by appropriately changing the enthalpy of the gas at the final pressure. This method is inherently more accurate than the common method of calculating compression power by using an average isentropic temperature coefficient (and other average gas properties) and the pressure ratio.

The computer program used for these analyses incorporates the Starling and Han modification of the Benedict-Webb-Rubin equation of state. This equation of state provides high accuracy for all thermodynamic parameters of the gas.

The data along the route used for hydraulic analysis includes items such as elevation, diameter, wall thickness, roughness, ground temperature and conductivity, and depth of burial. If the pipe is insulated, that too can be modelled.

The main equations used for hydraulic modelling are presented in Figure 5.5.

The primary hydraulic analytical design problem is to obtain a workable (ideally, optimum) combination of the primary design parameters: flow rate (including throughput factor), diameter, pressure, station spacing and station power. Other hydraulic variables include roughness, wall thickness, and maximum and minimum temperatures. For the design of pipelines in permafrost, secondary parameters such as flowing gas temperature assume a larger role in overall design.

There are two phenomena related to deviations of real gases from ideal gas behaviour that can have a significant influence on pipeline design:

1. The supercompressibility factor, Z

   The benefits of increasing the density of the flowing gas in the pipeline have been noted above, primarily in terms of the behaviour of ideal gases. The Z factor gives a measure of density deviation and should be considered in conjunction with pressure (and to a lesser extent temperature) as a design variable. Z decreases strongly with increased pressure up to about 14000 kPa (at 10 °C) and rises thereafter. This effect considerably reinforces the benefits of increasing design pressure up to this inflection and detracts from it thereafter.

Momentum Equation (Integral form):

$$Q=\frac{\pi}{8}*\sqrt{\frac{R_{gas}}{M_{air}}*\left(\frac{T_b}{P_b}\right)}*\sqrt{\frac{1}{f}}*\sqrt{\frac{P^2_1-P^2_2-g_{local}*\left(\frac{M_{air}}{R_{gas}}\right)*G*\frac{P^2_{avg}}{Z_{avg}T_{avg}}*\Delta h}{G*L*T_{avg}*Z_{avg}}}*D^{2.5}$$

with friction factor

$$\sqrt{\frac{1}{f}} = 4*\log\left(\frac{3.7*D}{e}\right); \qquad \textit{rough pipe law.}$$

Continuity Equation:

$$\rho \; A \; V = \text{constant}.$$

Energy Equation (differential form):

$$C_p*(dT-JdP)+dh = S*K*(T_g - T_{gas})dx$$

with shape factor

$$S = \frac{2\pi}{\cosh^{-1}\left(\frac{2b}{D}\right)}.$$

Where:

Q	- Flow rate;	D	- Pipe internal diameter;
R_{gas}	- Universal gas constant;	L	- Pipe segment length;
M_{air}	- Molecular weight of air;	e	- Absolute pipe roughness;
G	- Specific gravity with respect to air;	ρ	- Gas dencity;
T_b	- Refrence standard temperature;	A	- Pipe cross-sectional area;
P_b	- Refrence standard pressure;	V	- Gas average velocity;
f	- Fanning friction factor;	C_p	- Gas specific heat;
P_1	- Upstream pressure;	J	- Joule - Thomson coefficient;
P_2	- Downstream pressure;	S	- Buried shape factor;
g_{local}	- Local acceleration of gravity;	K	- Ground thermal conductivity;
P_{avg}	- Average pressure;	T_g	- Ground temperature;
Z_{avg}	- Average compressibility factor;	x	- Distance along pipeline;
T_{avg}	- Average temperature;	b	- Depth of burial of pipe;
Δh	- Elevation change;	T_{gas}	- Flowing gas temperature.

Figure 5.5: Hydraulic equations

2. The Joule-Thomson coefficient

The Joule-Thomson coefficient is a measure of the cooling attributable to decreasing pressure. As discussed below this is the prime factor in staying within the "temperature window" of permissible flowing gas temperatures in permafrost. As such, it consequently becomes a major determinant of maximum station spacing in permafrost terrain.

In considering the interactions of the Joule-Thomson effect with design pressure, it is helpful to use a modified presentation of the effect rather than use the Joule-Thomson coefficient directly. Station power is nearly proportional to the excess pressure ratio above 1.0 rather than to the absolute pressure change. Unfortunately, the region where Joule-Thomson cooling has the largest proportionate effect nearly coincides with the pipeline design pressure of 9930 kPa.

For the hydraulic analysis for the proposed pipeline, flow rates, throughput factor, design pressure, and pipeline diameter have been specified. The hydraulic analysis is primarily concerned with station spacing and power. The limiting factor on station spacing can arise from either of two physical constraints in discontinuous permafrost zones, compression ratio and "temperature window". One of these constraints, compression ratio, is the more important in conventional terrain, although there are still some limits on flowing temperatures. The constraints and their rationale are discussed below:

1. Compression Ratio. The maximum compression ratio is usually limited by the maximum head (i.e., increase in specific enthalpy) that can be achieved using a single impeller in a centrifugal compressor. For typical configurations this limit on maximum compression ratio occurs at values in the range between 1.30 and 1.35.

   In principle, compressor stations can be designed with multiple impellers to achieve a higher compression ratio. The chief reason for limiting compression ratio is that the pipeline operates inefficiently at high pressure ratios and dissipates considerably more energy through frictional effects. The cause is the decrease in density and increase in flowing velocity as local pressure progressively drops further and further below the design pressure value downstream of each station. The magnitude of this effect is clearly visible in Figure 5.6 which shows that overall compression power increases considerably faster than directly proportionately to pressure ratio. It should be noted that the figure includes several simplifications and assumptions that preclude direct comparison of the detailed numerical values with the project hydraulics presented further below.

2. Temperature Window. In permafrost zones, the maximum and minimum flowing gas temperatures are set by geothermal analysis (in conjunction with stress analysis) to prevent excessive thaw settlement. Typical values for discontinuous permafrost regions are +10 °C and -1 °C.

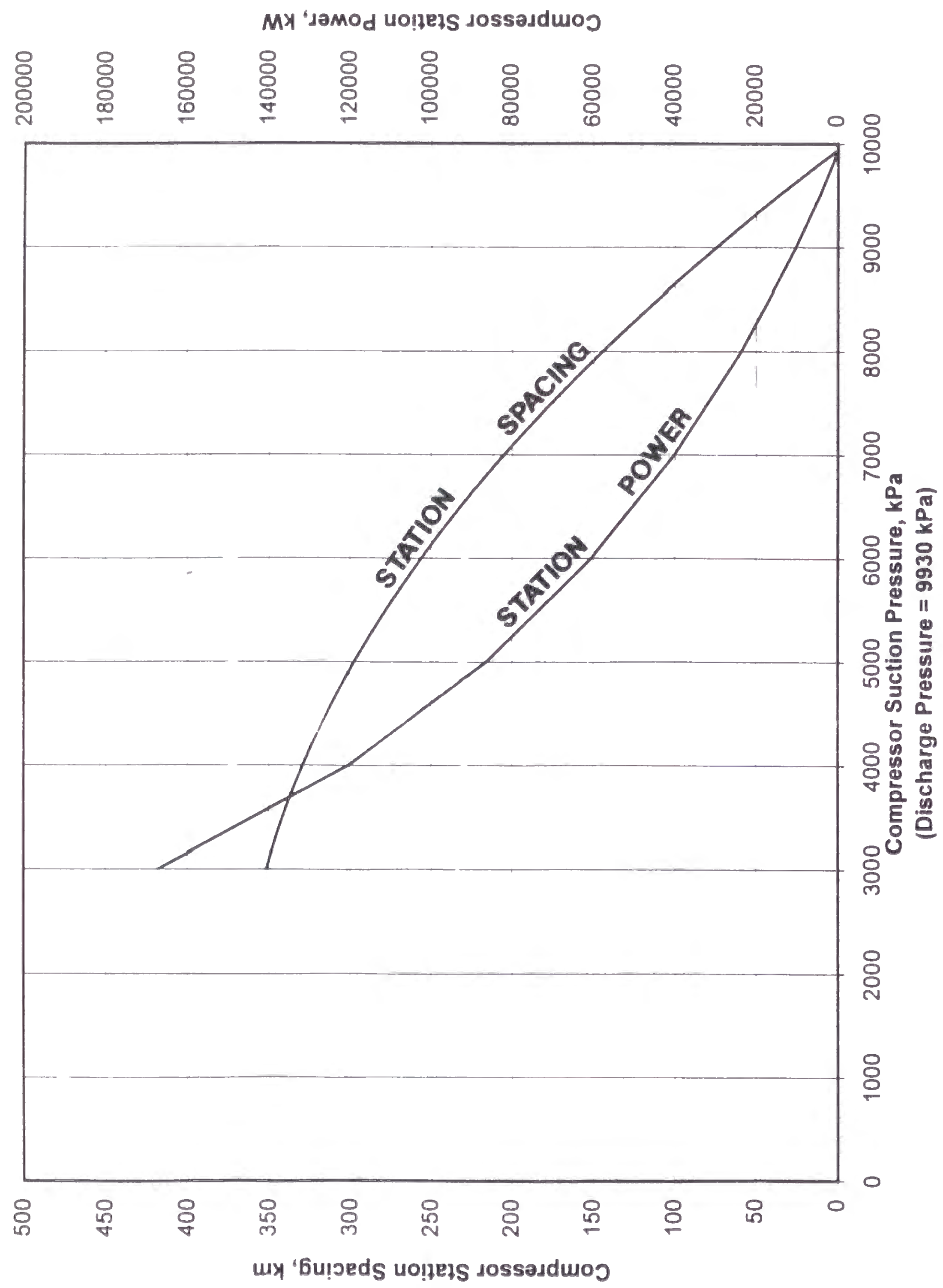


Figure 5.6: Compressor station spacing and power versus suction pressure

The temperature window combines with the effects of Joule-Thomson cooling and, to a much lesser degree, heat flux to the ground to set an upper limit on station spacing. An estimate of the pressure ratio corresponding to the Joule-Thomson effect that uses all the temperature window can be estimated from the gas properties. This estimate of the equivalent pressure ratio caused by the Joule-Thomson effect will be only slightly higher than if ground heat flux is also considered.

In conventional zones, the minimum temperature is not applicable (although a value far below freezing might induce frost heave even in conventional terrain). The maximum temperature is set either by:

- the maximum temperature for long term stability of the pipeline external (and internal) coatings

- the (unlikely) possibility that the pipeline could become unstable and buckle out of the ground in weak soils due to thermal expansion.

- the environmental damage that could occur from excessive heating, such as drying and killing plants above the pipeline.

Some points to note regarding the input data and assumptions in the hydraulic analyses include:

1. The value of pipe internal absolute roughness has been assumed as 0.05 mm which is near the upper limit for pipe that is not coated internally.

2. Station compression efficiency has been assumed as 80% for all stations under all operating conditions. Efficiencies of new compressors of modern design may be expected to exceed this value with peak efficiencies of 85% or more, but there will always be operating conditions that vary from the optimum efficiency point of the compressor head-flow map and there will be other losses, such as those due to station piping.

3. Because the computer program used for hydraulic analysis does not automatically account for fuel, an allowance for gas consumption has been made at the suction of each station to provide for the primary use of mainline compression, but also for chilling compressors, electric power generation, building heat and other auxiliary uses.

4. Representative ground temperatures and conductivities have been used with winter temperatures of 0°C and summer ground temperatures of 10°C. Sensitivity studies have confirmed that the hydraulics are not very sensitive to these parameters and further refinement of their exact values and variations along the route is not warranted for preliminary engineering.

The following comments are offered with respect to the output of the winter hydraulic analyses for a typical large-diameter long-distance pipeline:

1. The winter flow rates are the controlling design condition for station spacing because of the effect of the throughput factor.

2. In permafrost areas the limiting condition on station spacing is the temperature differential, although, for most stations, the resulting spacing is only slightly less than would be obtained from applying a maximum compression ratio in the 1.30 to 1.35 range.

3. The effects of elevation profile on pressure loss and corresponding Joule-Thomson cooling introduce some small variability in station spacing.

4. Chilling loads in winter at the permafrost stations in order to limit discharge temperature to 10 °C are quite high in comparison to compression loads (over 80% on an energy basis), indicating that little of the energy added by compression to overcome friction is being dissipated to the soil environment. Note that the chilling loads are the amount of heat that must be removed from the gas, not the power required of the chiller compressors.

The following additional comments are offered with respect to typical summer hydraulic analyses:

1. Compression power at all stations is roughly half of the winter design value for a typical throughput factor of 0.9.

2. Chilling loads are approximately 40% of the winter value. This is again because of the effect of the throughput factor.

Sensitivity analyses can be performed to assess whether the workable solutions are far from optimum and to understand better the interrelationships between the parameters, variables, and constraints. Three such sensitivity analyses are discussed below:

1. A series of hydraulic analyses was made for various flow rates, diameters, and station spacings at a constant design pressure and station compression ratio. Some simplifying assumptions were made to keep the analyses tractable, including isothermal flow (thus obviating the effect of temperature window constraints), constant flow rate along the pipeline (i.e., with no flow additions or takeoffs, including station fuel), and a throughput factor of 1.00. The relationships between flow, station spacing, and diameter were expressed as a series of power laws, using least squares statistical analysis based on a limited number of hydraulic runs.

   A simplified cost of transport spreadsheet was developed that permitted studying the relationships of hydraulic, cost, and financial variables in the form of a series of classic "J-curves." The cost of transport spreadsheet was supported by simple parametric forms for estimating capital cost and operating cost for each configuration. The cost estimate and cost of transport financial parameters could themselves be varied to examine sensitivities under various costing and financial scenarios.

   The spreadsheet is a dynamic instrument that can be used to quickly evaluate many alternatives. An example of one such analysis is presented on Figure 5.7. Some points to note:

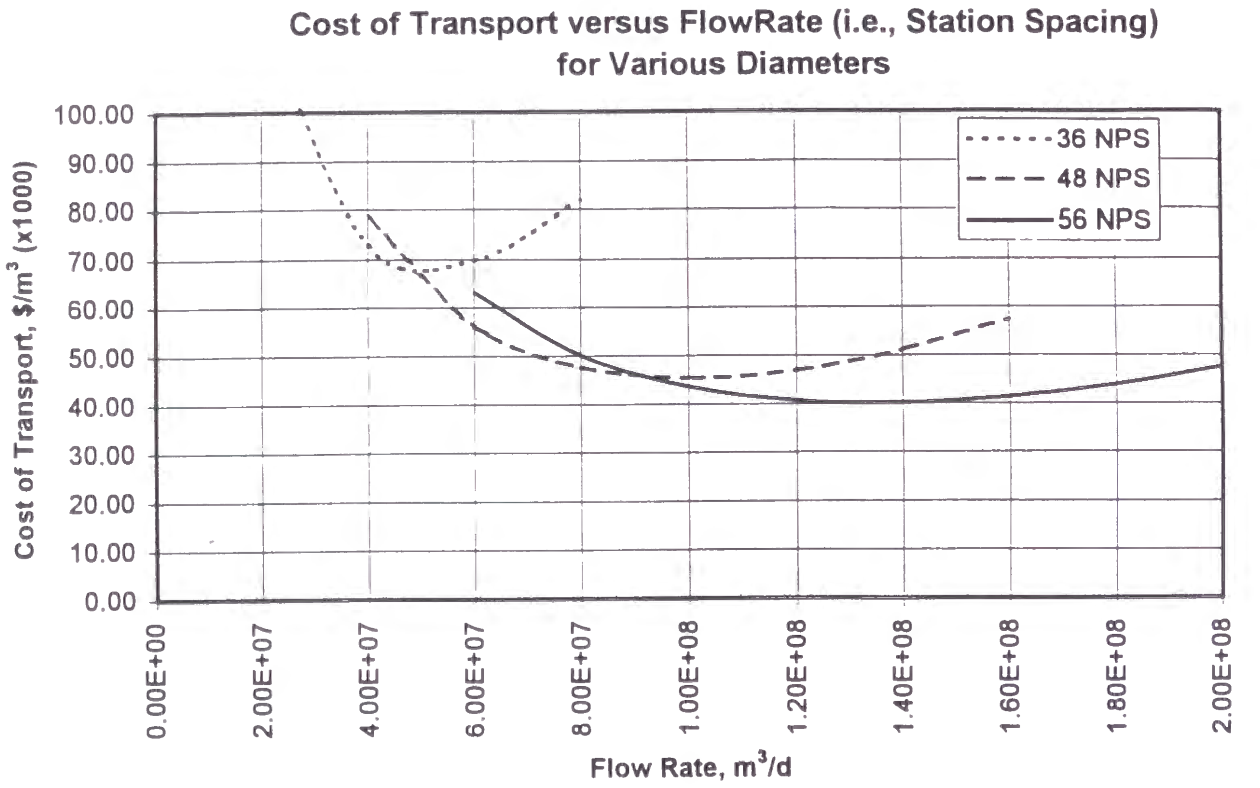
   - For each diameter there is a minimum cost of transport point associated with an optimum flow rate and station spacing. Further, as the diameter is increased, the cost of transport minimum becomes progressively lower but optimum flow increases.

   - For the larger pipe diameters, the individual J-curves tend to have a fairly broad valley over which the cost of transport is close to the minimum rather than a sharply defined notch. This validates the strategy of initially building a pipeline with fewer stations and subsequently adding stations to increase flow capacity.

   - The curves for various diameters intersect; each diameter has a range of flows (and associated station spacings) over which it is superior to other diameters. The envelope of all the curves is the limit of optimization (subject to the parameters and constraints kept constant to generate any given series of J-curves).

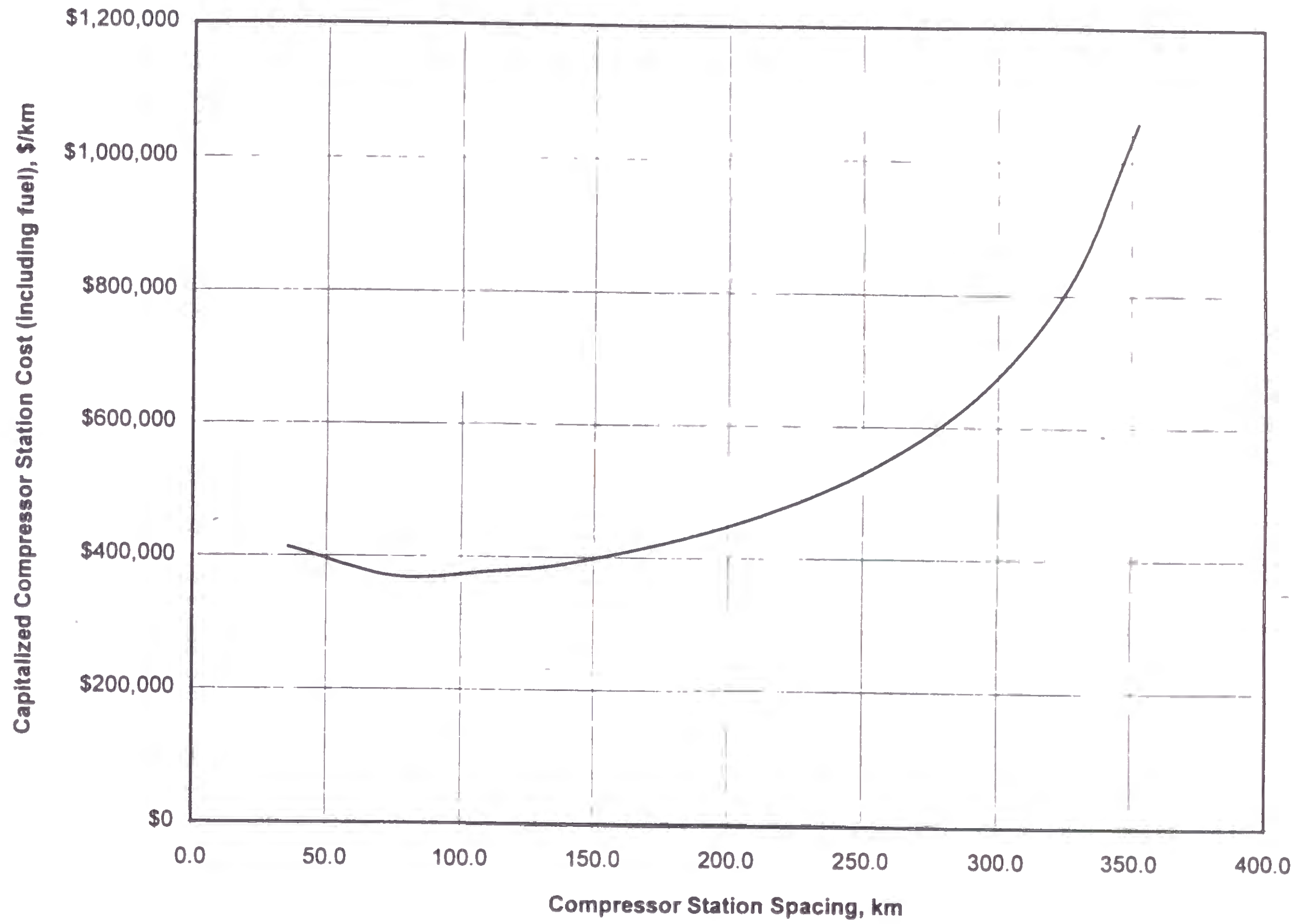
   - For many reasonable combinations of cost and financial parameters, the minimum for, say, 56 NPS and 48 NPS pipelines are only slightly different. This implies that there is only a small cost of transport penalty in choosing a diameter that is less than optimum. Conversely, it also suggests that a pipeline design diameter might be changed, for instance, to take advantage of particularly attractive commercial terms from some pipe supplier who was limited in the size of pipe that could be produced.

2. A sensitivity analysis was performed to understand whether the compressor station spacing criteria of single impeller (i.e., compression ratio less than 1.35), and a temperature window of +10°C to -1°C, were imposing a significant economic penalty on the pipeline. The method used a simplified discounted cost approach on a computerized spreadsheet. The power and spacing relationships were obtained using the basis previously presented in Figure 5.6. Capital costs were modelled as a simple multilinear function of station compression power and chilling load. Operating costs were modelled primarily in terms of fuel gas cost and compressor driver fuel consumption pattern. The chief financial parameters were a discount rate and time period. The spreadsheet is dynamic and can model a wide range of scenarios. The results of one such analysis are presented on Figure 5.8.



pipe length	2900	km
pipeline cost	$45,000	$/dia-in-km
stn $const	$0	station fixed cost
stn $/kw	$2,000	station compression power cost (100% spare)
$/mcf	$1.00	fuel cost = $35.31 $/m3
stn eff	30.00%	station driver thermal efficiency
o&m %	2.50%	percent of capital cost (pipe & stns) also includes property tax
debt interest rate	10.00%	
return on equity %	15.00%	after tax
debt/total capital %	75.00%	
income tax rate	50.00%	percent of before tax return on equity
discount rate	15.00%	use debt rate x debt/total capital + equity rate/(1 - income tax rate) x equity/total capital
life	20	years
payment %	15.98%	annual cost as % of capital to retire capital over life at discount rate (no tax deferrals)

Figure 5.7: Cost of Transportation versus flow rate



station fixed cost	5.00E+06	$
station variable cost	500	$/kW
capitalized fuel cost	1,702.71	$/kW

fuel cost	2.00	$/mcf
station thermal efficiency	30%	
annual fuel cost	200	$/kW.y
discount factor	10%	
life	20	y
npv factor	8.51	

Figure 5.8: Capitalized compressor station cost versus station spacing

Some points to note:

- For reasonable combinations of the cost estimate and financial parameters the optimum spacings do not exceed the spacings imposed by the compression ratio and temperature window criteria. In fact, if the fixed term of the multilinear cost estimate function is small, the optimum station spacing becomes very short, implying a very large number of stations.

- For reasonable combinations of the parameters the discounted costs are fairly flat for low compression ratios but increase rapidly for higher ratios.

3. A sensitivity analysis was performed to examine the effect of pipe roughness on station spacing. Roughness was reduced to 0.005 mm corresponding to a very high quality internal epoxy coating on the pipe. This reduced the friction factor considerably and allowed station spacing to increase by 15 to 20 km. The size of this benefit must be weighed against the fairly pessimistic estimate of the roughness of uncoated pipe which was taken as 0.05 mm. The question of internal coating should be readdressed during detail design.

5.3.4 Line Temperature Profiles

Based on the requirements to limit the station outlet temperatures in the permafrost region, the line temperatures will range between +10°C and -1°C. South of the permafrost region, the pipe temperatures can be allowed to warm up, however, cooling is eventually required to control the maximum temperatures to the recommended design limit of 40°C. Figure 5.9 shows the temperature profile for the entire pipeline system, based on the hydraulic analyses.

5.3.5 Geothermal Analyses

A direct threat to the structural integrity of the pipeline may occur from differential thaw settlement. This can occur where there is a transition from soils with high ice content, which are highly susceptible to thaw settlement, to soils that are not susceptible to settlement. Large relative displacements of the pipeline are possible if it longitudinally traverses such a transition. The effects on the pipeline are dependent on a variety of factors, including:

1. The abruptness of the transition
2. The amount of relative displacement
3. Soil properties, such as stiffness and strength
4. Pipe properties, such as wall thickness, and general configuration, such as depth of burial.

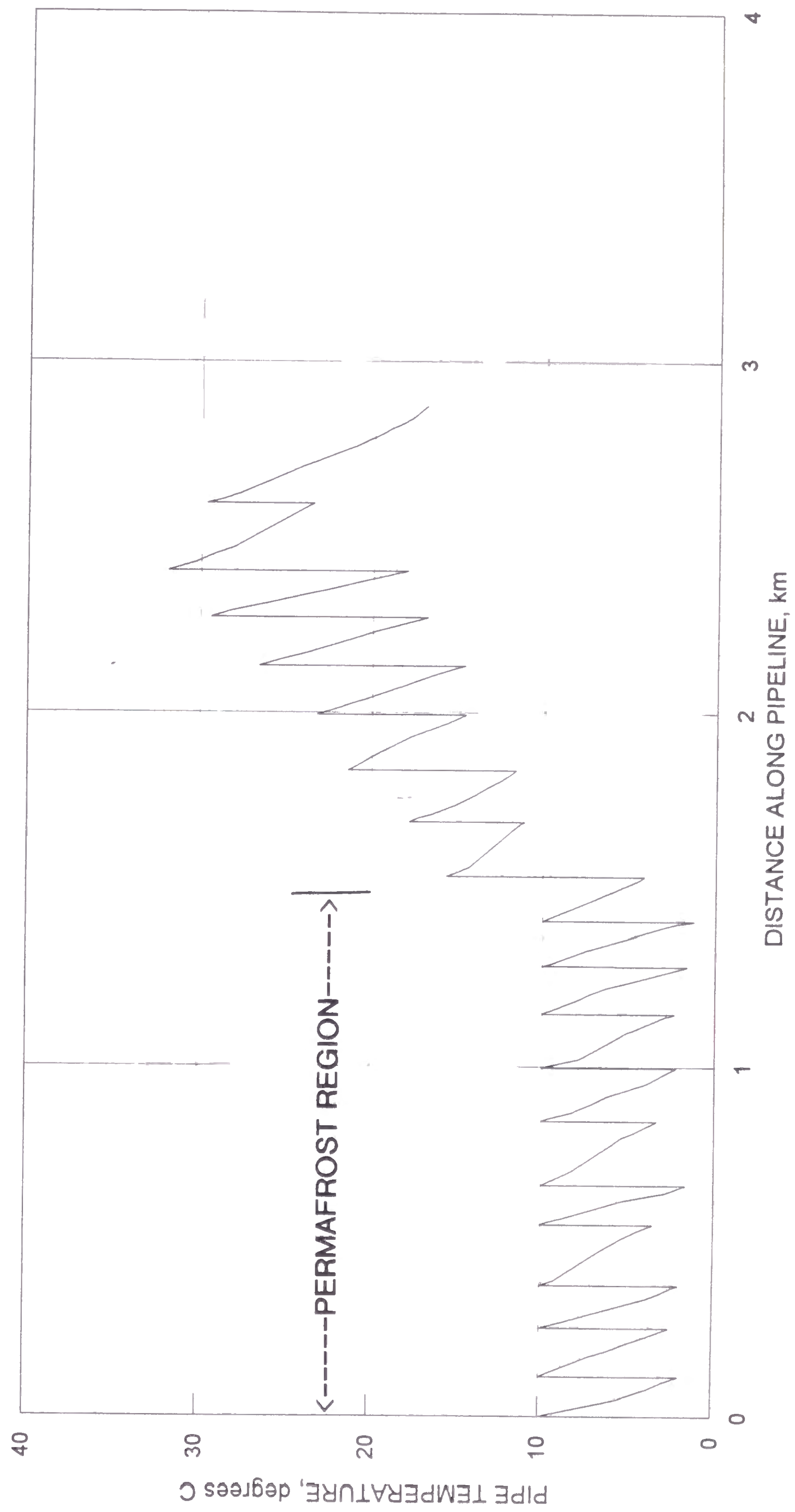


Figure 5.9: Line temperature profile

Predictions have been made for the thaw progression and the resulting thaw settlement. Physical and thermal properties of the typical frozen soil of the Angaro-Lenskiy geocryological region were taken for simulation of permafrost thaw and settlement under a pipe. Conservative parameters were adopted for a silty clay - total moisture content of 30% and bulk density of 1 650 kg/m^3. The thaw settlement rate for this soil is 0.15. Typical thermal properties selected for the soil (heat capacity and thermal conductivity) are presented in Table 5.1. Winter in this region is seven months long. The average winter air temperature is -15.7°C. The average summer air temperature is 12°C.

TABLE 5.1 Thermal Properties of Selected Soil

Heat Capacity, kcal/m^3°C		Thermal Conductivity, kcal/m h°C	
Frozen	Unfrozen	Frozen	Unfrozen
550	650	1.6	1.4

The simulation has been conducted using a finite element simulator. An axi-symmetrical grid, was set up with the left border going through the vertical diameter of the buried 1 420 mm pipe. A mean annual surface temperature of 0.6°C was applied for 20 m from the pipe, representing the relatively warm, cleared right of way conditions. For the remainder of the surface, an undisturbed temperature of -1°C was applied. Gas temperatures of 40°C and 10°C were analyzed for a running time of 20 years.

Figure 5.10 shows the depth of thawing under the pipe and the adjacent area. The depth of thawing reaches about 15 m below the ground surface after 20 years of operation, with a gas temperature of 40°C, and about 10 m, with a gas temperature of 10°C. Figure 5.11 shows the thaw progression directly under the pipe. It can be seen that the thaw rate is maximum during the first seven to eight years, then it gradually decreases. Again, Figure 5.11 shows that the thaw rate is more gradual for gas temperature 10°C.

5.3.6 Thaw Settlement Potential

For the predicted thaw depths, the respective long term thaw settlement equals 1.87 and 1.1 m as shown on Figure 5.12. It should be noted that in the stress-strain analysis for the pipe, it is conservatively assumed that the full predicted thaw settlement could occur as differential settlement at any location.

The thaw settlement of 1.87 m is considered unacceptable with respect to pipeline stability. Consequently, it is recommended to cool the gas to an annual mean temperature of 10°C. The settlement of about 1.1 m is considered acceptable for a North American, strain-based pipe structural design. Therefore, it can be concluded that cooling the gas to a mean annual 10°C will permit a conventional buried design in the permafrost sections of the pipeline.

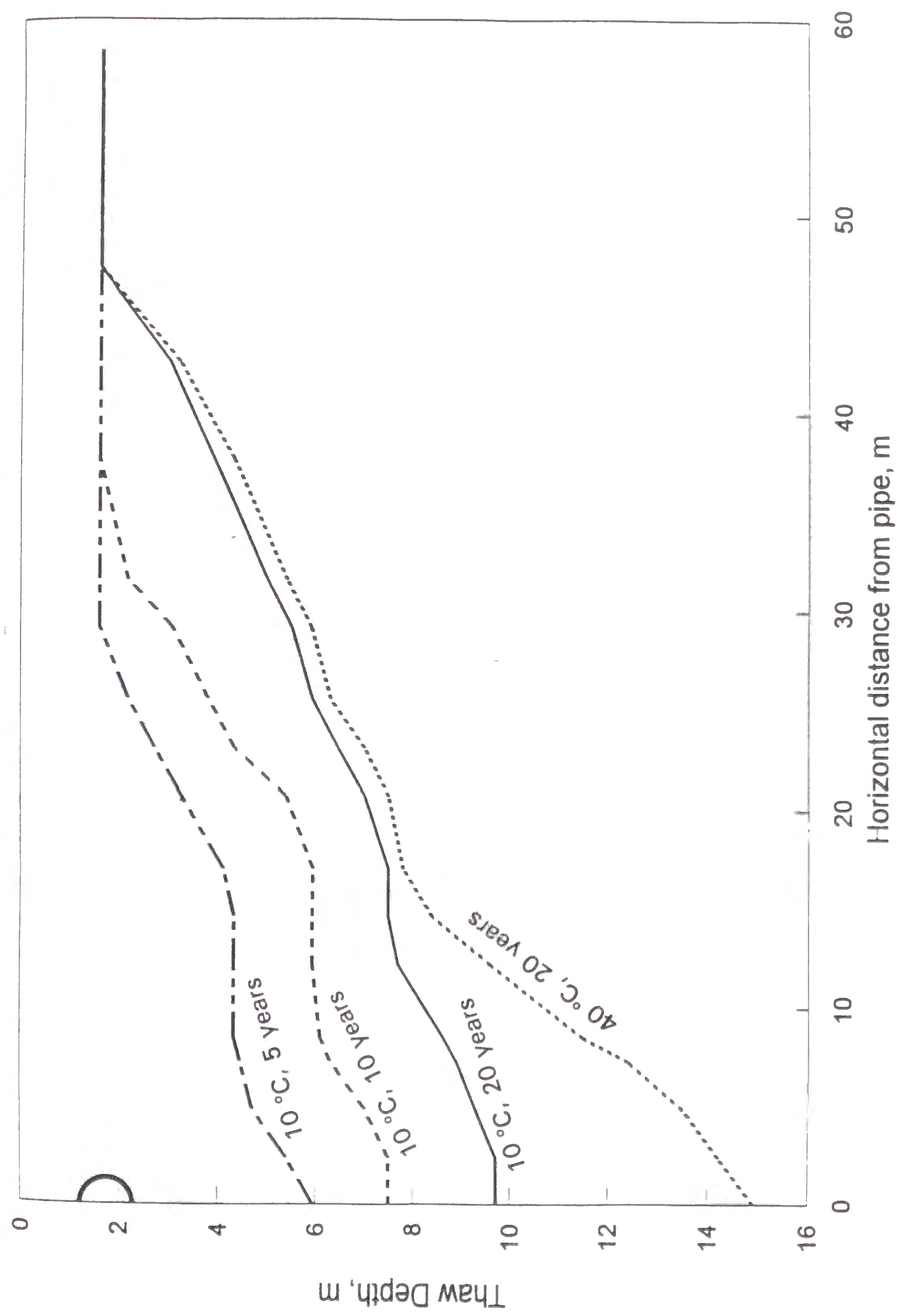


Figure 5.10: Thaw progress under pipe and adjacent area

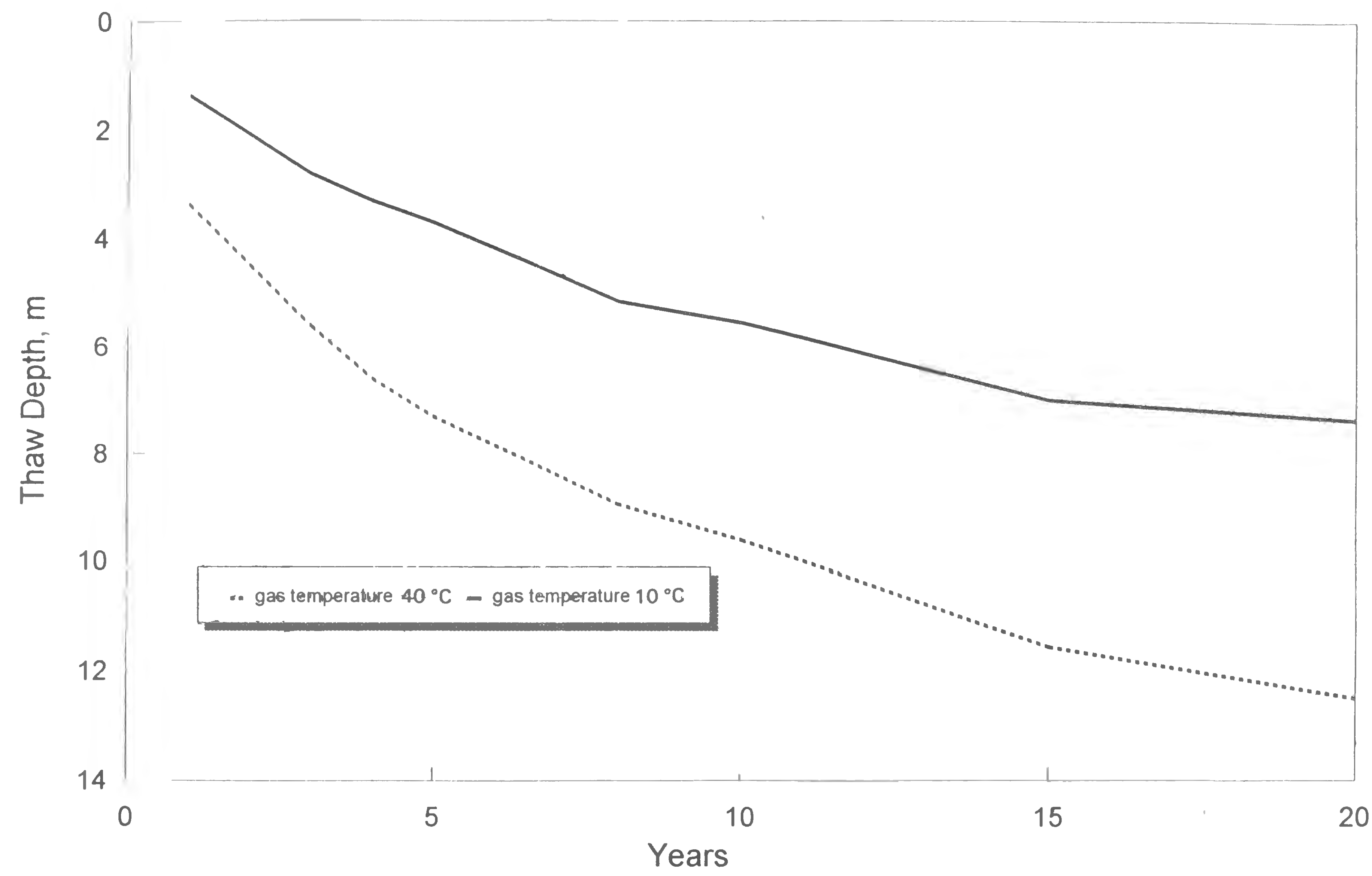


Figure 5.11: Thaw progression below base of pipe

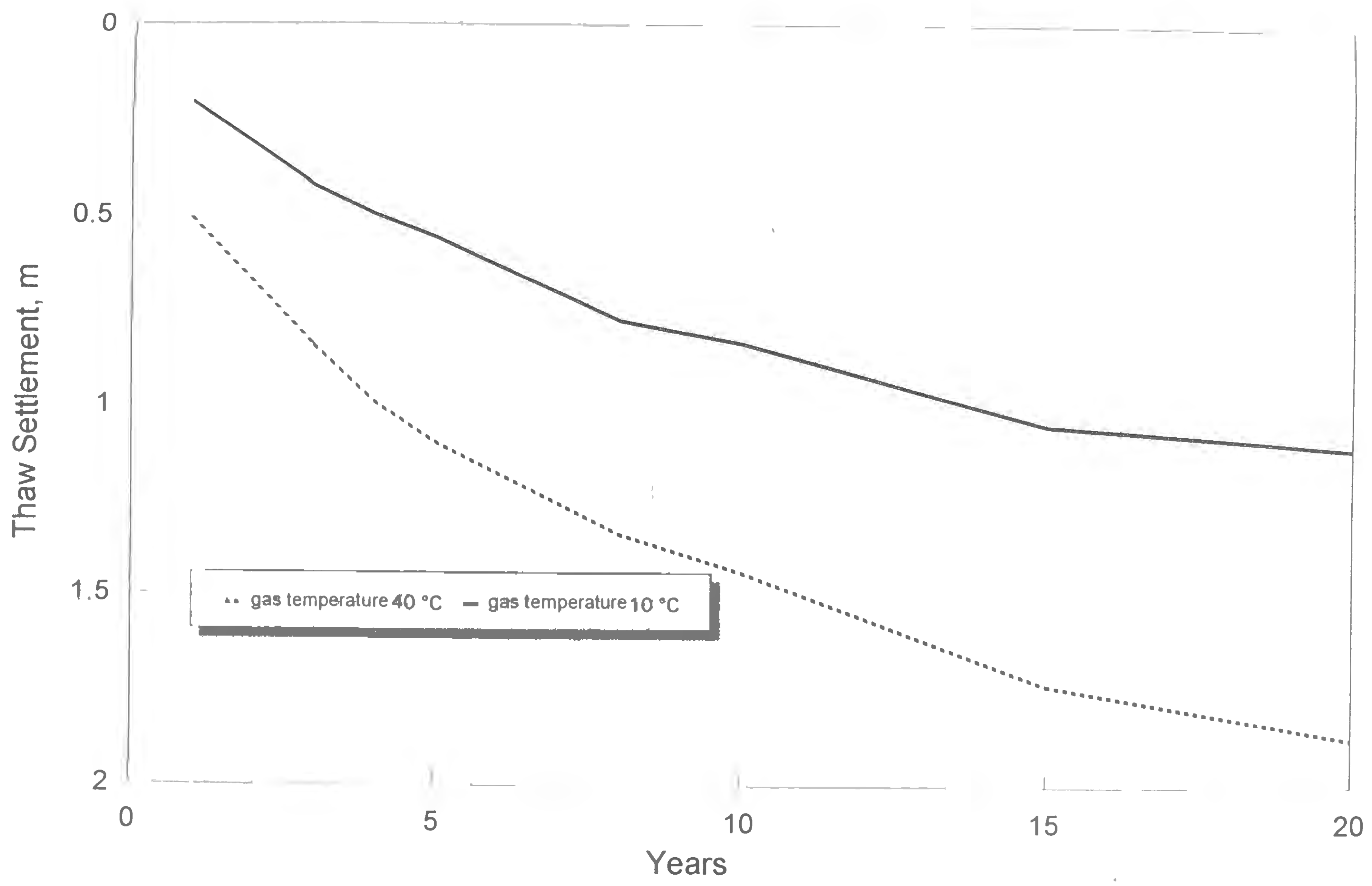


Figure 5.12: Thaw settlement progression below base of pipe

The processes of geothermal modelling, stress-strain analysis, hydraulics, and general mechanical design are interactive and iterative. The primary output of the geothermal modelling for further engineering are:

1. Maximum permissible pipeline operating temperature. Because of Joule-Thomson effects in the flowing gas, this is equivalent to maximum permissible compressor discharge temperature.

2. Expected near-maximum settlement corresponding to the maximum permissible temperature. This settlement amount must satisfy several subsidiary conditions to avoid environmental damage and to ensure pipeline integrity and the viability of an interventive maintenance strategy:

   - Settlements greater than the near-maximum must occur over only a small portion of the route in order to keep the scale of interventive maintenance manageable.

   - The near-maximum settlement and differential settlement must be readily detectable and must occur sufficiently slowly and progressively that interventive maintenance is practical.

After geothermal modelling, geotechnical engineering provides input to stress analysis with respect to the differential thaw settlement model and the soil properties. Model inputs include:

1. Transition lengths and transition soil and settlement functions between thaw-susceptible and non-thaw-susceptible soils, and single-sided versus double-sided (spanning) transitions and their length

2. Soil stiffness, strength, weight and related parameters.

5.3.7 Pipe-soil Interaction (thaw settlement)

The thaw settlement resulting from the pipeline construction and operation will be most severe closest to the compressor stations, where the warmest pipe temperatures will be experienced, as illustrated in Figure 5.9. The pipe-soil interaction that requires structural analysis is illustrated on Figure 5.13. The analysis conservatively assumes that the soil settles away from the pipe on the settling side of the transition. Thus the base of the pipe is no longer supported, and the pipe will deform due to its self weight and the loading from the backfill on top of the pipe. On the thaw stable side of the transition, Figure 5.13, the pipe is supported by the stable soil and bending is resisted by the stiffness of the pipe and to some extent by the loading from the backfill on the stable side.

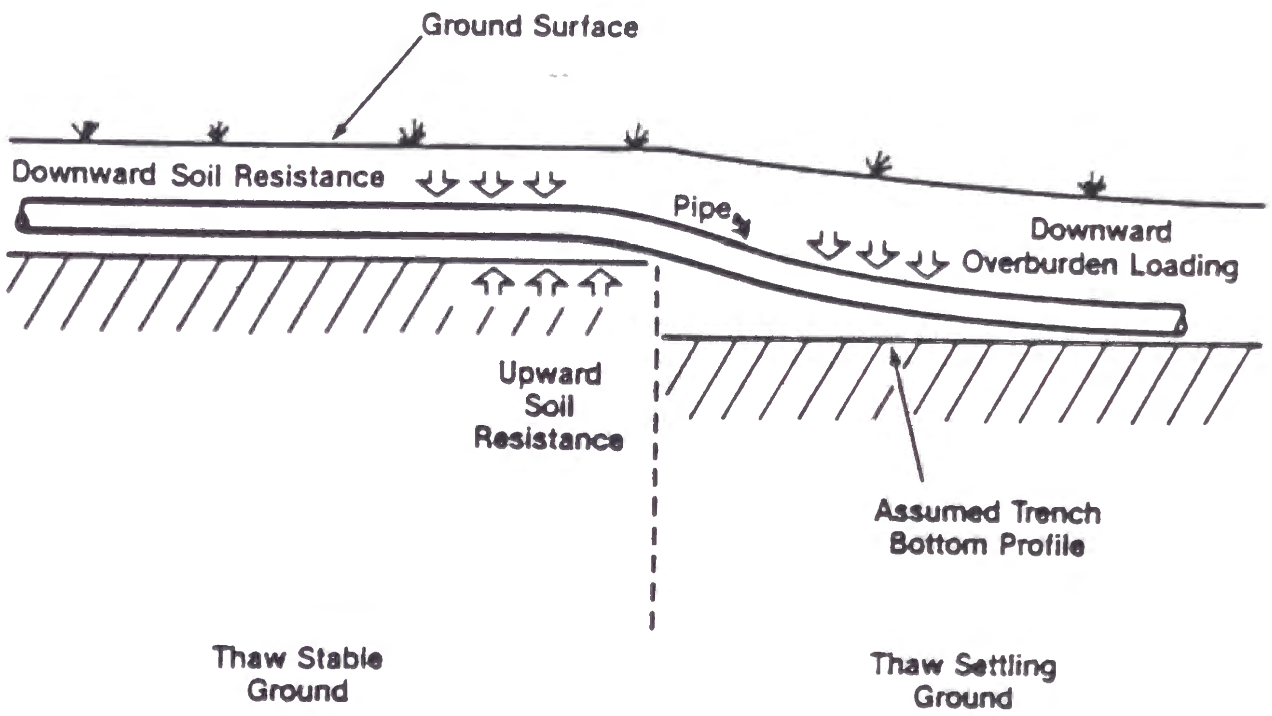


Figure 5.13: Differential thaw settlement at a transition

Apart for the stiffness of the pipe, it can be seen that the bending in the pipe will be related to the strength of the soil beneath the stable pipe and the load from the backfill on the settling side. The influence of the backfill loading will be directly related to the depth of cover over the pipe. Figure 5.14 illustrates (not the ESFE case) the fact that the allowable thaw settlement is reduced, if the pipe is buried deeper. This is a very interesting situation, as one means of reducing the potential thaw settlement is to bury the pipe deeper so as to avoid the typically more icy soils near the surface. This is a typical example of the "iterative" nature of the pipeline design process for permafrost conditions.

It is most common to assume that the settlement occurs over a large distance on the settling side of the transition. This might be considered the most conservative case. Obviously, if the length of the settling soil was very small (such as 10 m), the influence on the pipe would be minimal. It is of interest to examine the effect on the pipe stresses, as the length of the settling soil is increased. Figure 5.15 shows an example of such an analysis (not the ESFE case). It can be seen that there is actually a particular length of settling zone that experiences greater stresses that for a long settling zone. This needs to be considered in final pipe-soil interaction design.

5.3.8 Pipe Structural Analysis

The 1422 mm OD gas pipeline is expected to be installed in both thaw stable soil and in zones of thawing soil. The thaw settlement which will occur within the zone of thawing soil will induce a differential displacement in the pipeline at the transition from thaw stable soil to thawing soil. Soil-pipe interaction analyses were performed to determine the maximum differential thaw settlement the proposed pipeline can be subjected to in combination with the specified operating conditions without disrupting the normal operation of the pipeline.

The following briefly describes the analyses carried out and the parameters investigated. The results obtained are summarized and presented in curves showing displacement and temperature induced strain variations to illustrate the effect of the parameters examined.

5.3.8.1 <u>Summary of Data Used for the Thaw Settlement Analyses</u>

The pipeline data presently made available and the assumptions made in the selection of the additional data required to perform the analyses, but not as yet available, are briefly summarized.

Operating Conditions

The following operating conditions have been specified:

- maximum operating pressure 9930 kPa
- maximum operating temperature +10°C
- minimum installation temperature -40°C

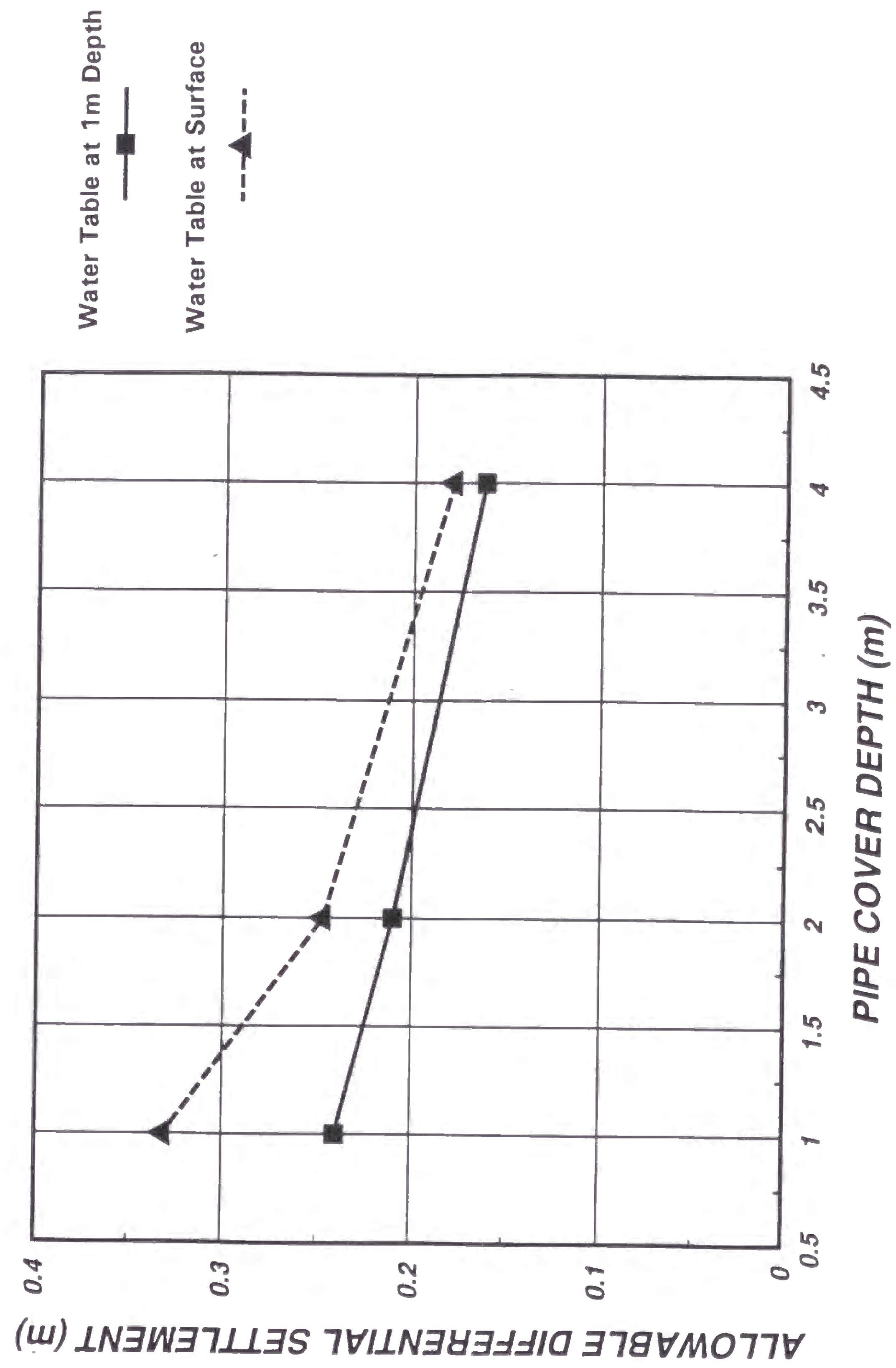


Figure 5.14: Illustration of the effect of pipe cover depth on allowable differential settlement

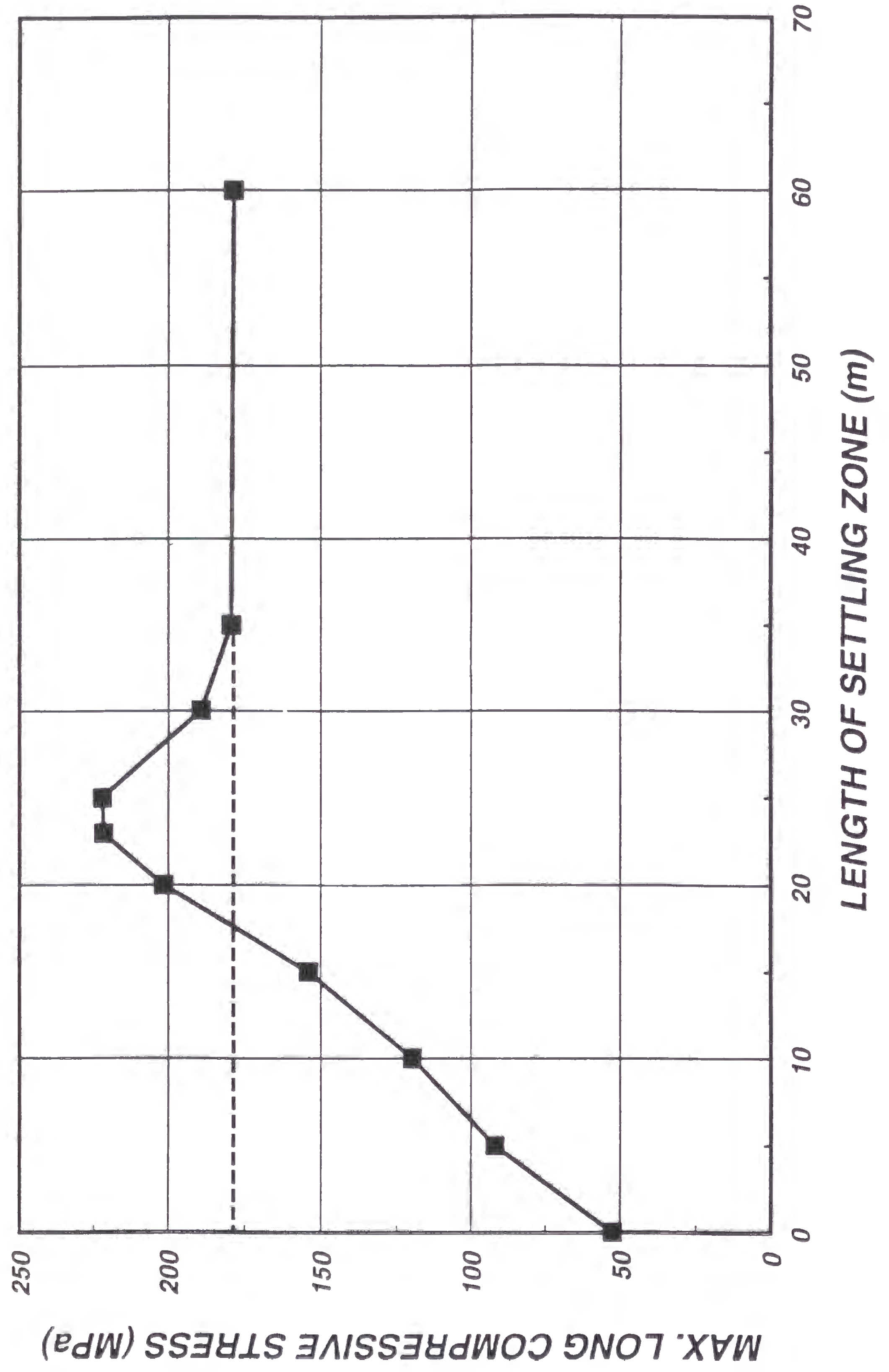


Figure 5.15: Illustration of the effect of finite length of settling zone on pipe stresses

Pipeline Data

The following pipe data is specified:

- outside diameter of pipe 1422 mm
- grade of steel API 5L X-65 (448 MPa)

It is proposed that the design of the pipeline will be specified to comply with the requirements of ASME B31.8. It is further proposed that cathodic protection or similar measures be used for corrosion protection of the pipe steel. Consequently, the selected pipe wall thickness does not need to include a thickness allowance for corrosion. Additionally, it is assumed that the specified pipe will qualify for a joint factor of 1.0. On this basis, application of the criterion for pressure containment specified by ASME B31.8 with a design factor of 0.72, results in an allowable wall thickness of 22.2 mm for the general cross-country line pipe.

It is expected that segments of the proposed pipeline will be fully restrained in the axial direction for the specified operating conditions. It is required, therefore, that the stress state in the pipe wall due to the specified operating pressure and temperature differential satisfy the combined membrane stress criterion for fully restrained pipe. A 22.2 mm wall thickness will be subjected to a combined stress state equivalent to 86% of the maximum allowable equivalent tensile stress in the fully restrained condition for the specified operating conditions. The 22.2 mm wall thickness is therefore also adequate for the fully restrained condition.

The following pipe specification was therefore used for the thaw settlement analyses:

- 1422 mm OD x 22.2 mm W.T., API 5L X-65

5.3.8.2 Analytical Method

An initially straight, buried pipeline subjected to thaw induced differential settlement will typically undergo large displacements and experience significant curvatures due to bending, with bending strains in the nonlinear range. Similarly, the soil surrounding the pipeline within the zone of differential settlement will exhibit nonlinear behaviour.

The principal damage considerations are fracture initiation due to high tensile strains and local buckling due to excessive compressive strains. In view of the displacement controlled nature of the thaw settlement induced pipeline deformations, strain criteria are used to define the limits for fracture initiation and initiation of local buckling.

Due to the nonlinear nature of the thaw induced differential settlement imposed on a buried pipeline, a nonlinear method of analysis is required to properly asses the response of the pipeline to the imposed loading. The computer program PIPLIN III was developed specifically for the nonlinear analysis of buried pipelines and has the capability of dealing with both large displacement and material nonlinearity. PIPLIN III was therefore used for the thaw settlement analysis carried out.

The analytical model consists of an initially straight pipeline extending past the virtual anchor point to a distance of 480 m on each side of the 14.5 m transition zone between thaw stable soil and thawing soil. The pipeline is idealized as an assemblage of pipe elements supported at each nodal point by discrete longitudinal and transverse spring elements representing the soil-pipe interaction. The material behaviour of the pipe steel is represented by a piece-wise linear approximation of the Ramberg-Osgood formulation of the stress-strain curve used in the past to represent API 5L X-65 pipe steel. The nonlinear behaviour of the soil is taken into account by assigning the bilinear characteristics to the spring elements.

The following series of thaw settlement analyses were carried out using the analytical model described above.

- The steps of the analysis were to apply the pressure loading of 9930 kPa, followed by the temperature differential of +50°C and, as the last step, apply the displacement on the thaw settlement profile in increments until the maximum settlement is reached. With the assumption that the ground water level is at the mid-point of the pipe, the pipeline is essentially in a neutrally buoyant state. The gravity loading due to the weight of pipe, pipe coating, etc. was therefore not included in the analyses.

- The response of a nonlinear system to loading is dependent on the sequence in which the loading is applied. An analysis was therefore carried out to assess the effect of load path dependence by reversing the application of the temperature differential and the thaw settlement profile.

- An analysis was carried out with an abrupt step settlement profile to assess the influence of the geometry of the thaw settlement profile within the transition zone. The base case load sequence was used.

- Three analyses were carried out to examine the sensitivity of the axial compression strain to variations in the ultimate resistance of the thaw stable bearing capacity springs. The analyses were carried out using a 25% reduction, a 25% increase and 50% increase in ultimate resistance compared to the base case resistance.

5.3.8.3 Summary of Results

Base Case Analysis

This analysis was carried out to establish the maximum differential settlement the proposed pipeline can be subject to in combination with the specified operating conditions and differential thaw settlement due to the presence of the 50°C positive temperature differential. Based on Lagner's expression for the compressive strain associated with initiation of local buckling, an expression commonly used for this purpose, it can be expected that local buckling may initiate at a compressive strain level of 0.78%.

The base case analysis was therefore carried out with the thaw settlement profile applied in increments until the maximum longitudinal compressive strain was found to exceed 0.78%. This resulted in a maximum differential settlement of 1.52 m and corresponding maximum axial compressive and tensile strains of 0.85% and 0.35%, respectively.

An allowable axial compressive strain of 0.55% and an allowable axial tensile strain of 0.55% have been proposed for displacement controlled loading.

The variation of the maximum axial compressive strain and maximum axial tensile strain with increasing thaw displacement is shown in Figure 5.16. Also shown are the proposed allowable compressive strain and allowable tensile strain limits. It is seen that the tensile strains are not critical, whereas the allowable compressive strain limit will be exceeded for a thaw displacement of approximately 1.1 m. Consequently, the maximum allowable thaw displacement should be limited to around 1.1 m if the maximum allowable compressive strain is to be limited to 0.55%.

For the maximum thaw settlement case, Figure 5.17 shows a plot of the thaw settlement profile and the profile of the displaced pipeline base at the transition zone. It is observed that the pipeline does not have contact with the ditch base for most of the length of the transition zone and for some distance beyond the transition zone. That is, for the assumed transition settlement profile and a thaw settlement of 1.52 m, the pipeline is unsupported for a distance of over 30 m.

In this condition the pipeline is subjected to a primary load system from the overburden soil, pipe weight, etc. in addition to the thaw settlement induced displacements. The integrity of the pipeline must in this case be assessed in terms of primary load based stress criteria in addition to the displacement based strain criteria discussed above. In view of the small cover depth relative to the pipe diameter it is not expected that the primary loading will be the critical factor for the maximum allowable thaw settlement, but this is a condition which must be assessed in the final design.

The variation of the axial strain in the top and in the bottom of the pipe along the length of the pipeline in the vicinity of the transition zone is shown in Figure 5.18 for the maximum thaw displacement of 1.52 m. Both the maximum compressive strain and the maximum tensile strain are seen to occur at the start of the transition zone adjacent to the thaw stable soil zone.

Analysis to Assess Influence of Load Sequence

The results obtained from the analysis carried out with reversal of the application of the temperature differential and the thaw settlement profile are shown in Figure 5.19 in terms of the variation of the compressive strain and the tensile strain with increasing values of positive temperature differential. The results are shown for the maximum thaw settlement of 1.524 m. The maximum compressive strain and tensile strain obtained in the base case analysis for the same level of loading is -0.854% and 0.353%.

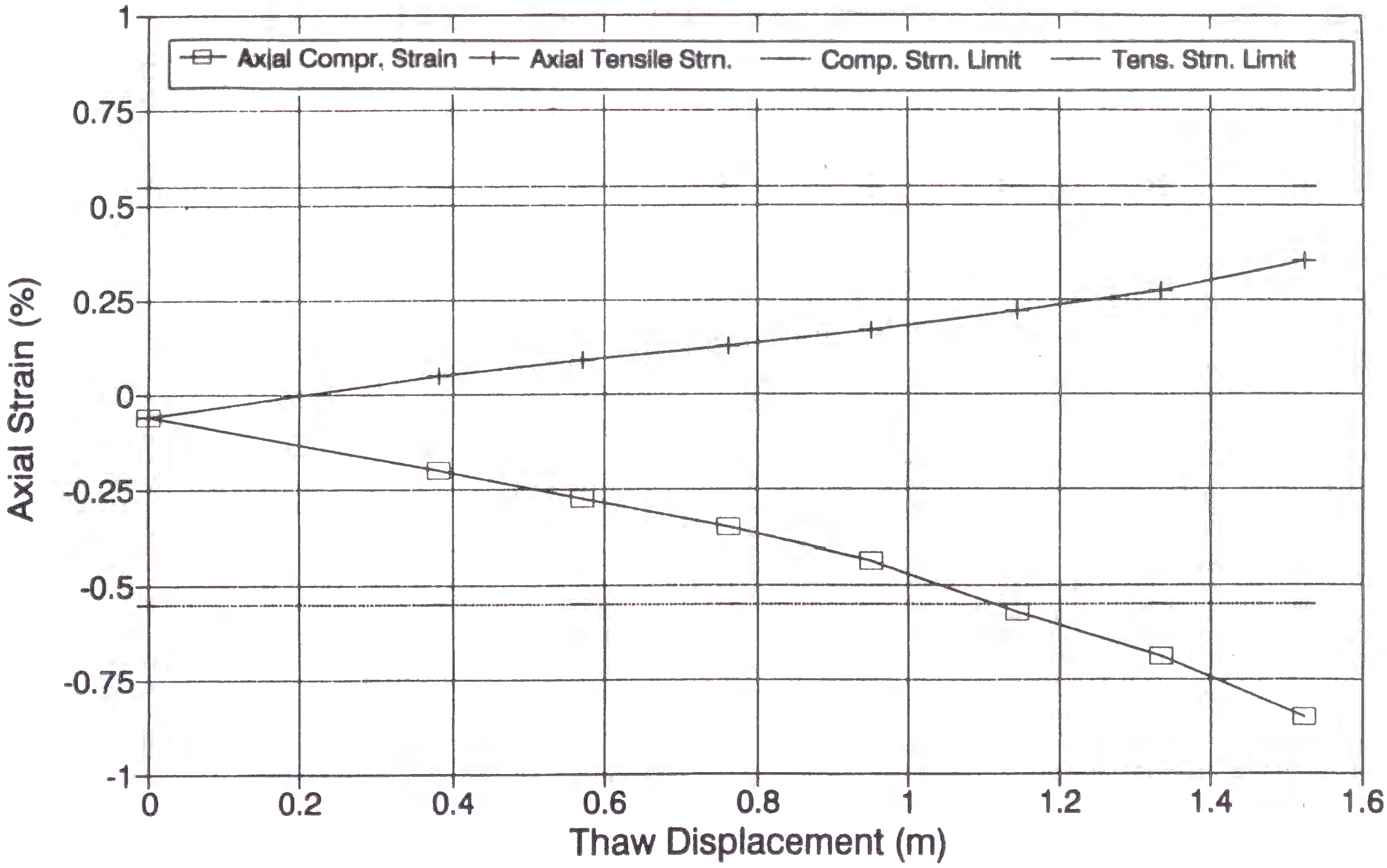


Figure 5.16: Axial strain versus thaw displacement at location of maximum axial strain

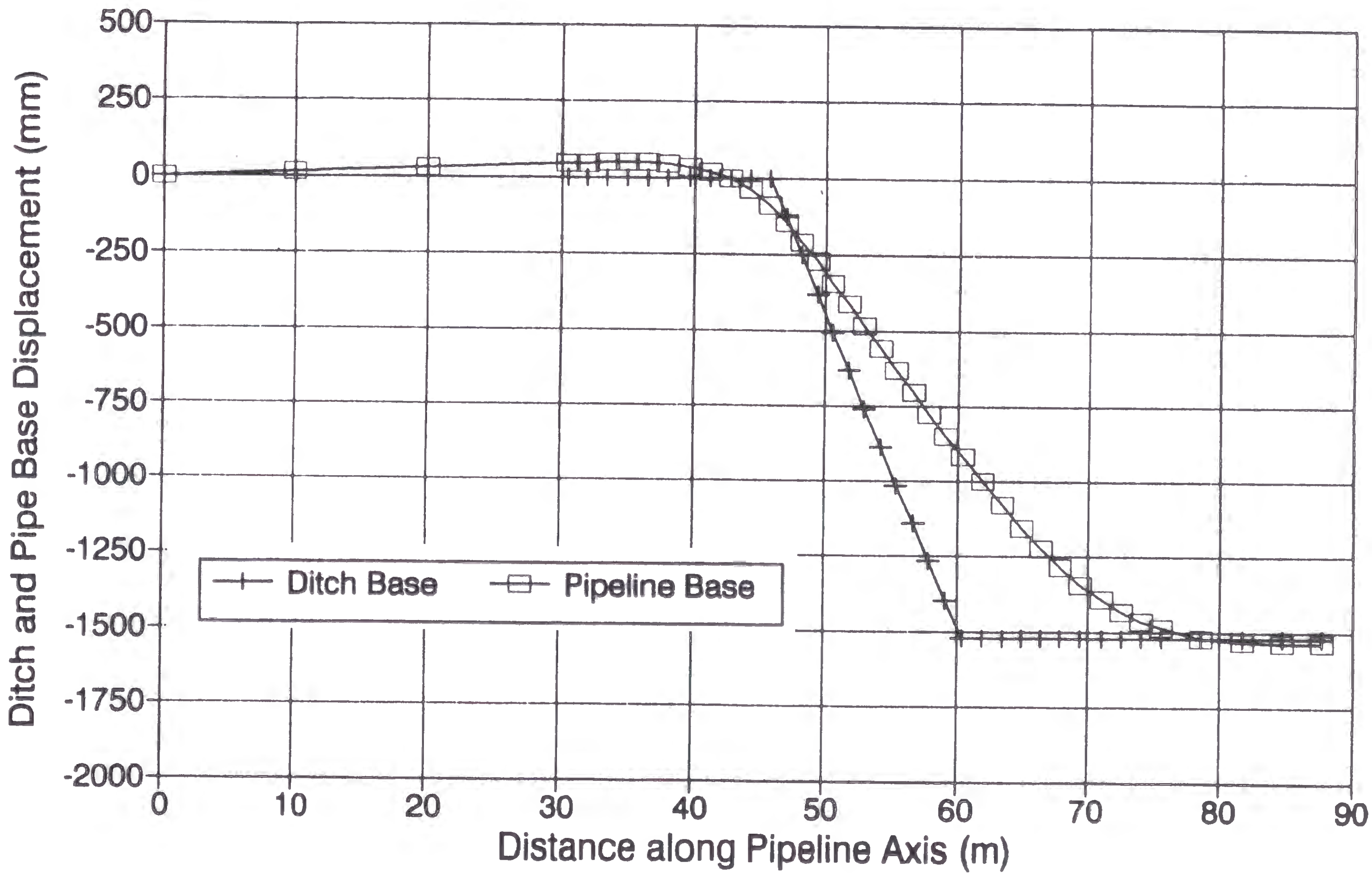


Figure 5.17: Profile of ditch and pipe displacement at thaw settlement interface

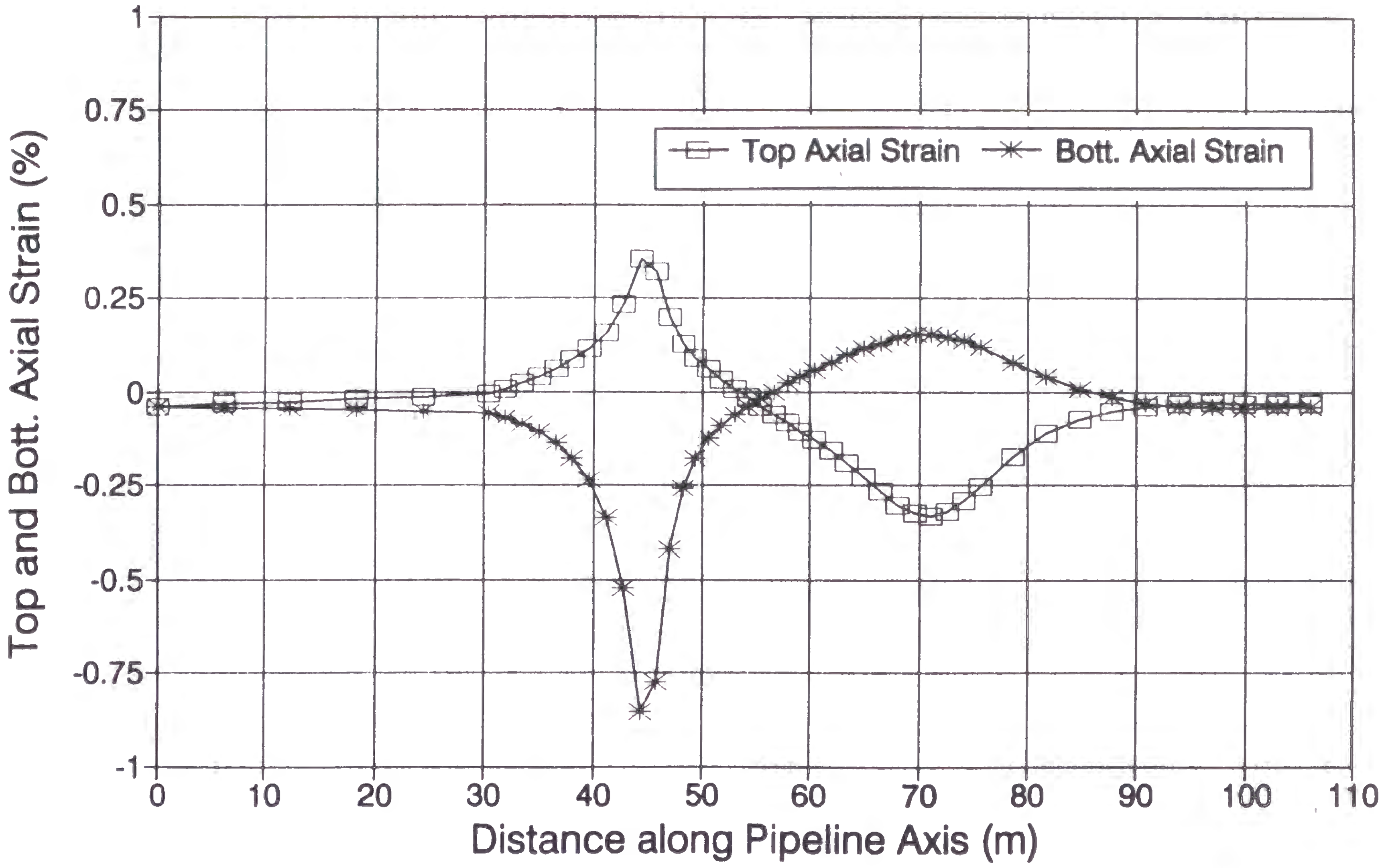


Figure 5.18: Axial strain variation along pipe at thaw settlement interface

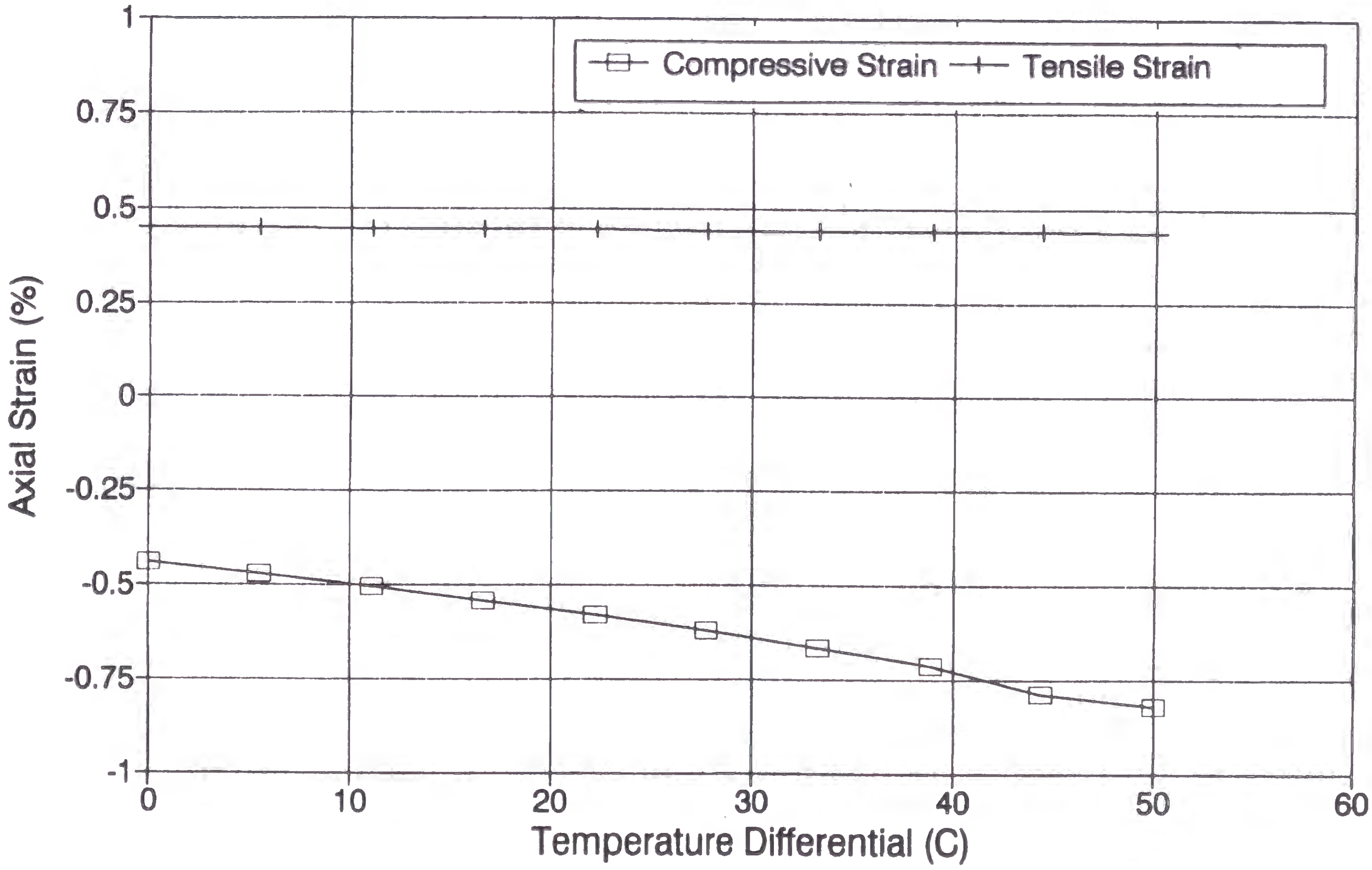


Figure 5.19: Axial strain versus temperature differential with 1524mm thaw settlement

The compressive strain level is seen to depend significantly on the level of temperature differential imposed, whereas the tensile strain level shows little variation with changing temperature differential. A reduction of the imposed temperature differential to around 20°C would allow the thaw settlement to increase to 1.5 m for the proposed maximum allowable compressive strain criterion of 0.55%.

Analysis to Assess Influence of Thaw Settlement Profile

The compressive strain results obtained from the analysis using an abrupt step profile for the thaw settlement profile are compared in Figure 5.20 with the compressive strain results obtained from the base case analysis, which used the assumed linear ramp profile to represent the thaw settlement. The step profile is seen to result in higher compressive strains for a given thaw settlement amount. Consequently, the geometry of the thaw settlement transition zone can be expected to have a significant impact on the maximum thaw settlement displacement allowed.

Effect of Variations in Bearing Capacity of Thaw Stable Soil

The compressive strain results obtained from the various analyses carried out using different ultimate resistance values for the thaw stable bearing springs are shown in Figure 5.21 as a function of thaw displacement. In general it is observed that the compressive strain level is relatively insensitive to a change in the bearing capacity of the thaw stable soil. A 50% increase in the bearing capacity of the thaw stable soil resulted in only a 7% increase in maximum compressive strain for the maximum thaw settlement of 1.524 m.

5.3.9 Acceptable Temperature Differential

As indicated in the discussion of the pipe stress results, the temperature differential between the "construction" temperature and the warmest operating temperature, is a critical parameter for the pipe structural design. This relates to the compressive stresses that develop in the pipe due to the thermal coefficient of expansion. In winter construction conditions, the temperature of the pipe may be as cold as -40°C. While winter air temperatures colder than -40°C can exist, it is not normal for construction work to be carried out in colder temperatures. The maximum operating temperatures could be as high as +40°C, giving rise to a potential temperature differential, or "delta T", of 80°C.

For the proposed pipeline, it has been established that the maximum operating temperature in the permafrost region will be 10°C. Hence, the maximum design delta T is 50 C. Figure 5.22 provides an illustration of the influence that delta T has on the allowable thaw settlement (not the ESFE case).

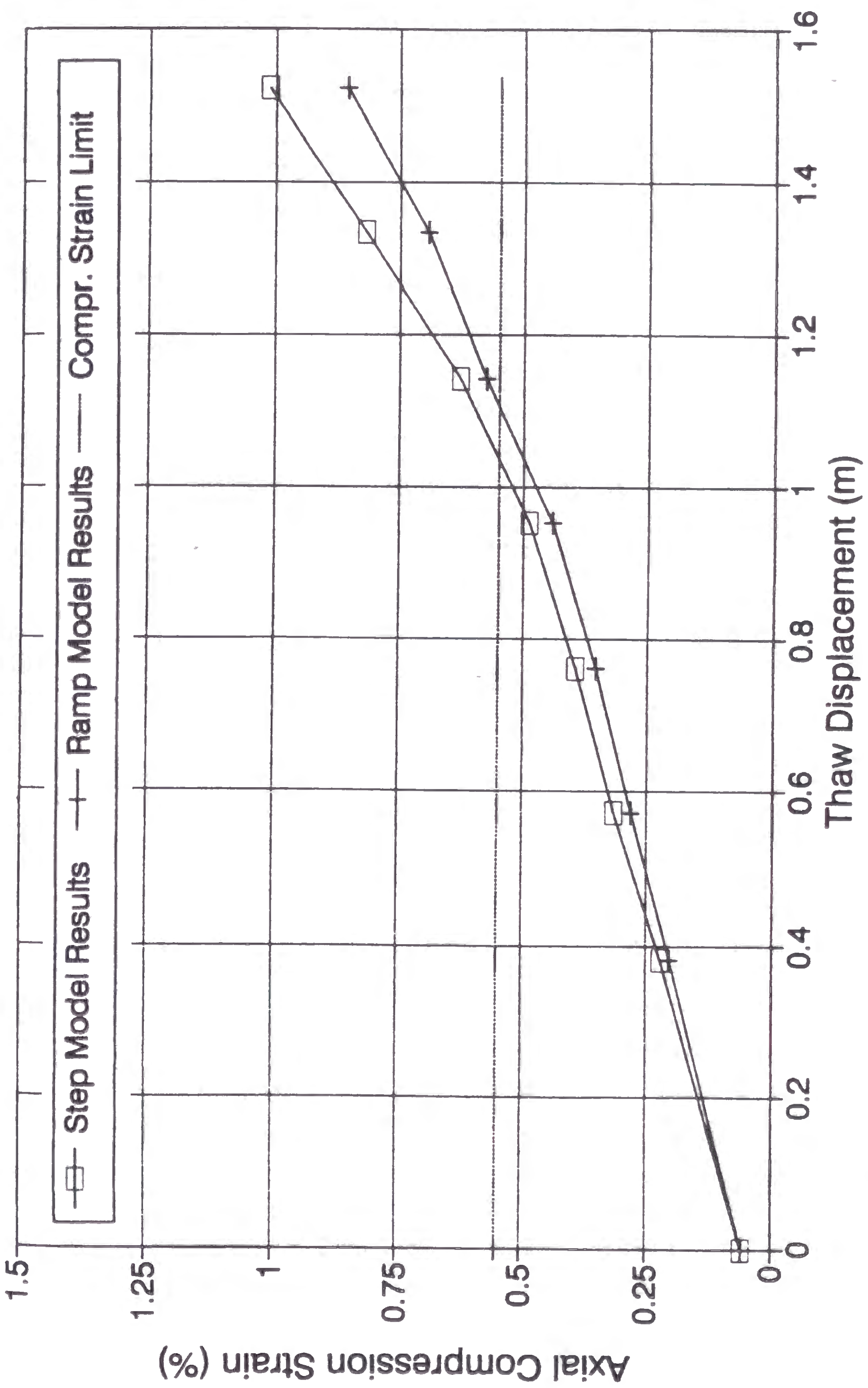


Figure 5.20: Compressive axial strain versus thaw displacement comparison of ramp and step model results

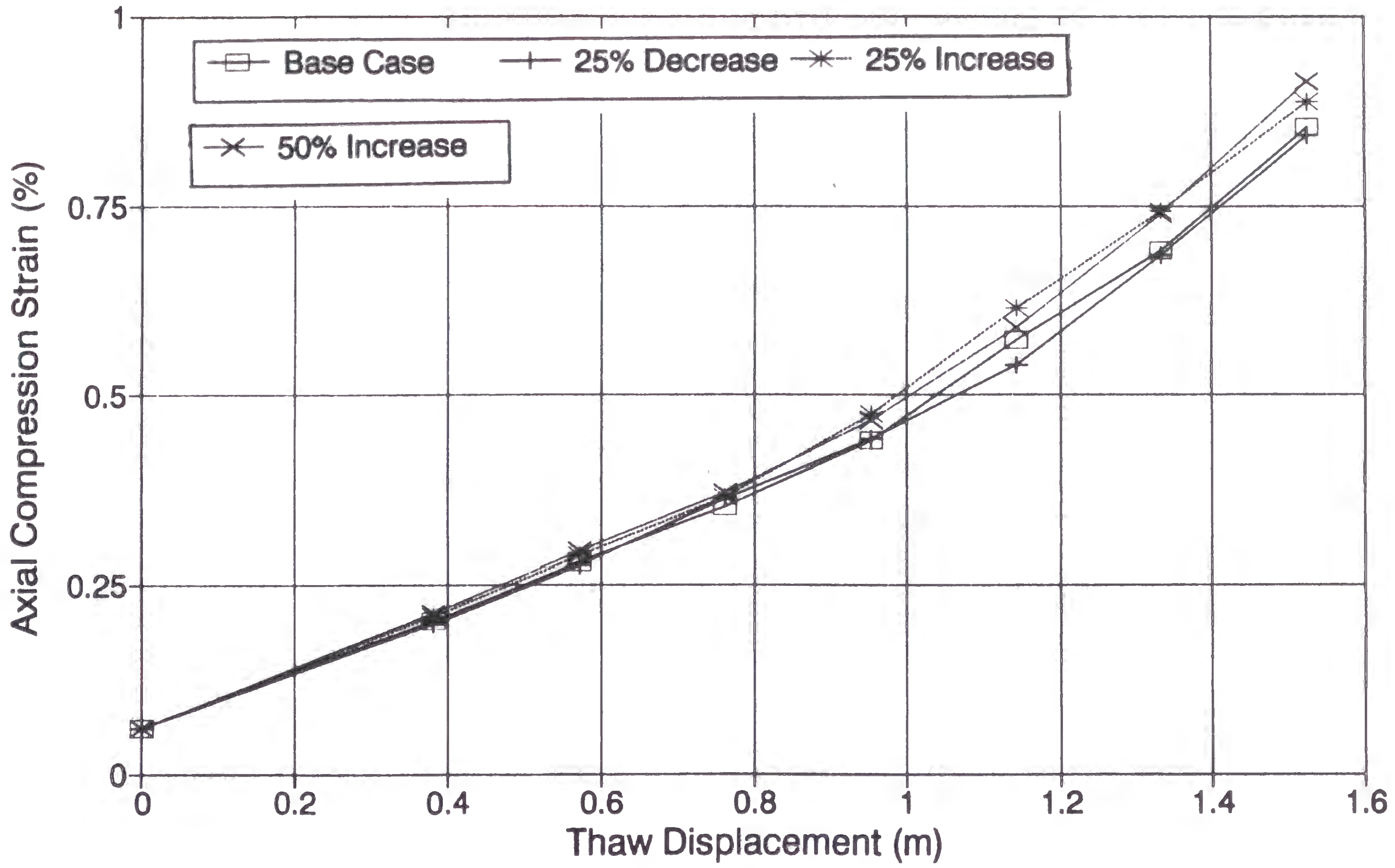


Figure 5.21: Compressive strain versus thaw displacement effect of the thaw stable bearing strength

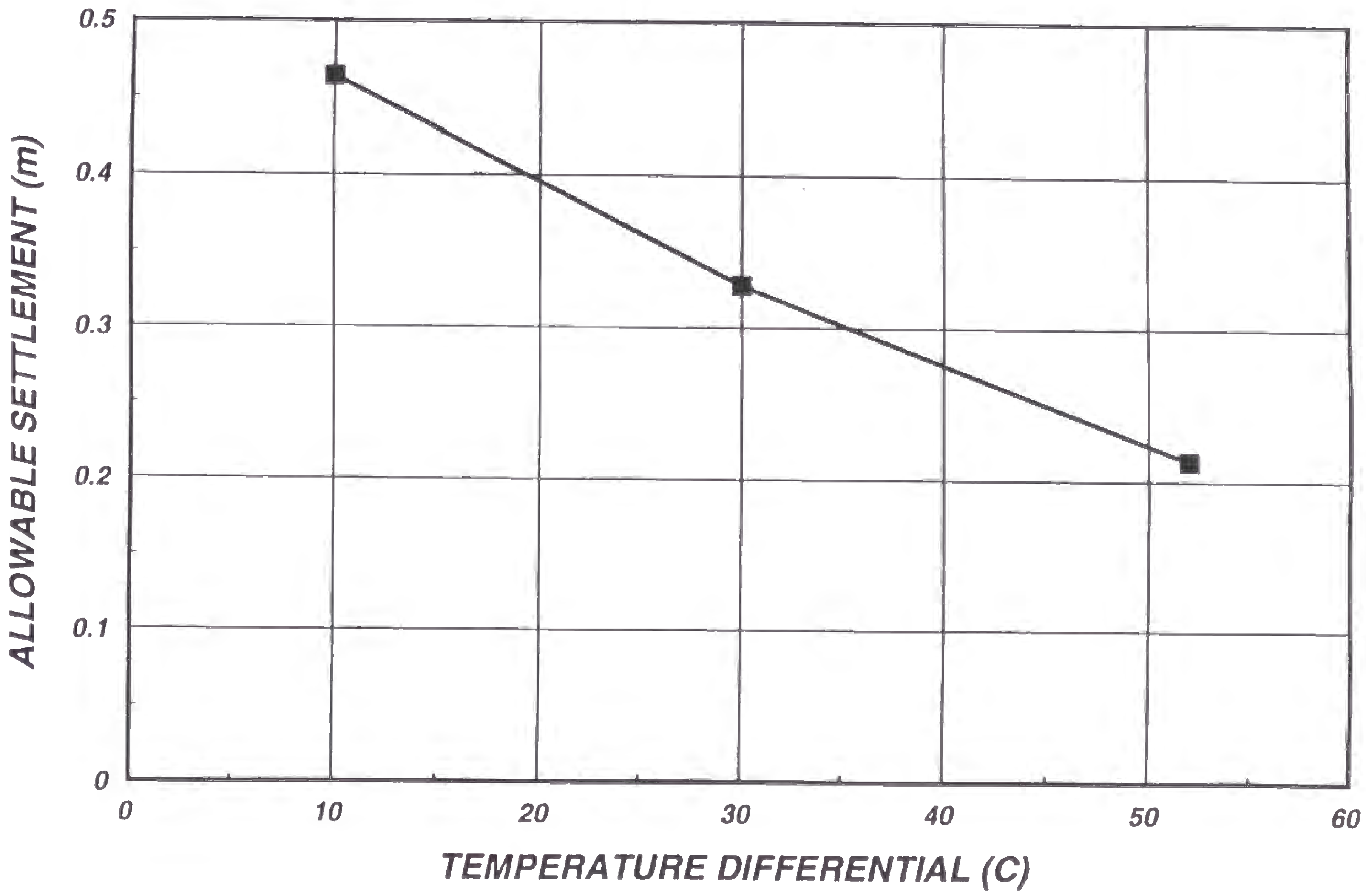


Figure 5.22: Illustration of allowable settlement as a function of temperature differential

5.3.10 Permafrost Slope Stability

The design for the thawing slopes along the pipeline will be based in the approached presented in Section 4.3.6. For the soil parameters available from the Ust'-Kut region, all ice rich slopes steeper than 9° will require some form of slopes mitigation. The mitigation will include:

- minimizing width of clearing and surface disturbance
- construction of ditch plugs
- placement of granular backfill
- insulation of the pipe, or above grade mode
- placement of wood chips
- cutting steeper slopes to a more stable, lower angle
- construction of drainage and erosion control berms
- seeding and fertilizing

The unique aspects of the thawing permafrost slopes design procedure are presented. This approach was developed specifically for the major North American pipeline designs being undertaken in the 1970's and 1980's (McRoberts and Morgenstern, 1974; Hanna and McRoberts, 1988).

5.3.10.1 Geothermal Effects

One component of the design analysis is to establish the slope angle below which no mitigative measures are required. In order to undertake such analyses the dimensions of the thaw bulb with time are required. It is important that sufficient insulation be placed on the pipe to ensure the pipe does not cause more rapid thawing than would otherwise occur. In some more sensitive cases, the pipe may have to be constructed above-grade to limit heat input to the ground.

Natural, Non-Insulated Surfaces

The thaw penetration versus time relationships used in the analyses are presented in Figure 5.23. It is assumed that a considerable depth of thaw may occur during the first thaw season. The maximum depth of thaw of 6.1 m for ice rich soils is constrained by the empirical observations relating to a 13 m wide right of way which is recommended as a desirable limit to the width of right of way.

Insulated Surface

It is possible to reduce the anticipated depth of thaw by placing an insulating layer on the natural ground surface. The type of insulation recommended is a natural insulating material such as wood chips. The assessment of insulation thickness requirements involves geothermal analyses with input parameters including air and ground surface temperatures, as well as the thermal properties of the various soil and insulating layers.

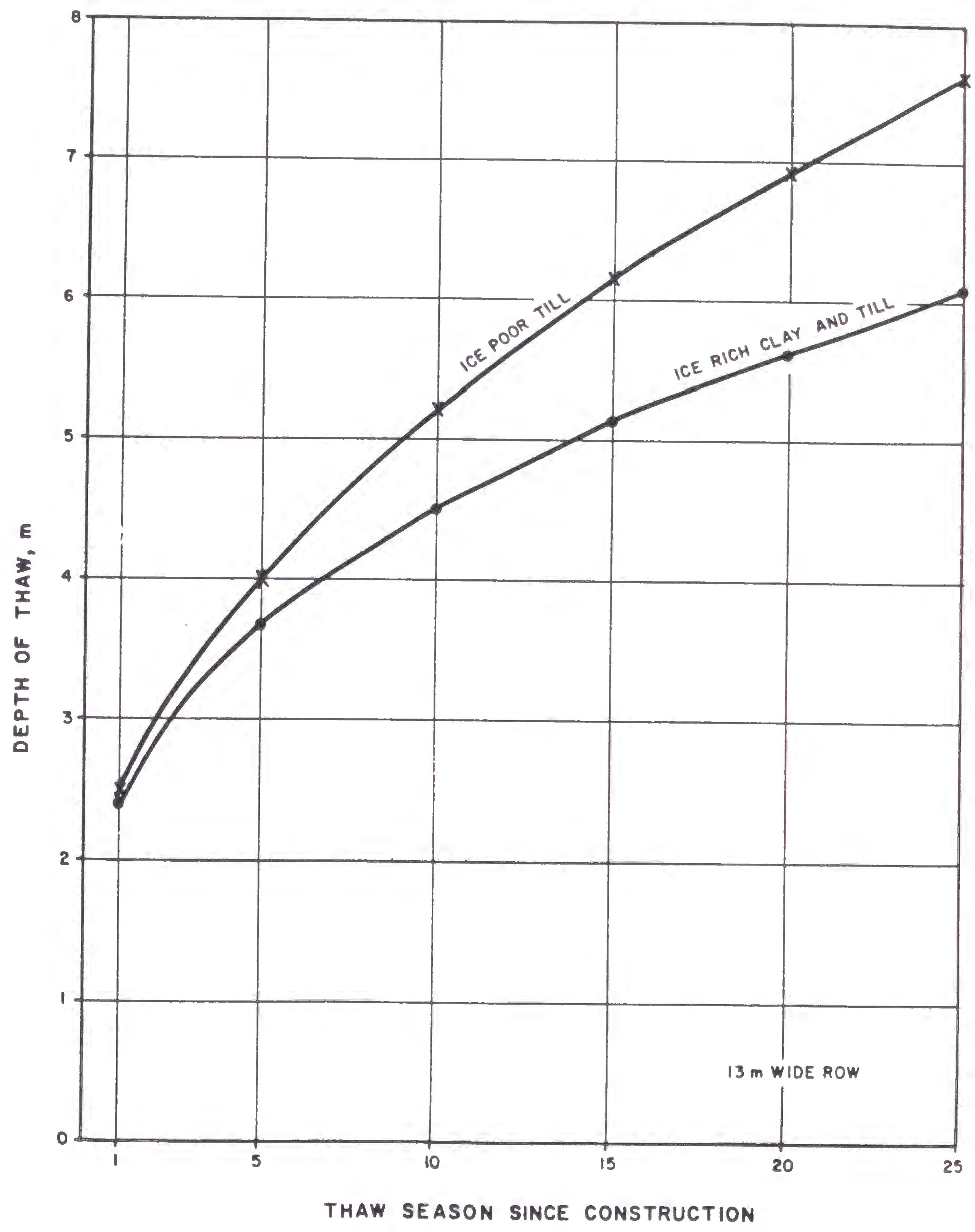


Figure 5.23: Depth of thaw versus thaw season used in analysis

The temperature of the ground responds primarily to the annual cycle of air temperature change at the ground surface. The ground surface temperatures are similar to the air temperatures, but are modified by the properties of the surface material, and the effect of snow cover during winter. The mean monthly temperatures for Irkutsk, given in Table 5.2, have been used. The wood chips are by themselves a good thermal insulator. Predictions show that a sufficient thickness of wood chips can essentially prevent thaw within the 25 year period following construction.

Pipe insulation should be used in cases where only a nominal amount of wood chip insulation is required and the pipe temperatures are close to zero degrees. For steeper or more ice rich slopes, where the pipe temperatures are warmer than 1 or 2°C, the pipe may have to be placed on the surface.

Table 5.2 Mean Monthly Air Temperatures, Irkutsk

Mo.	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
Temp	-20.9	-18.5	-10.0	0.6	8.1	14.5	17.5	15.0	8.0	0.1	-10.7	-18.7	-1.2

5.3.10.2 Excess Pore Pressures

In the first thaw season, thaw proceeds more or less in accordance with the form

$$d = \alpha\sqrt{t}$$

where:
- d is the depth of thaw
- α is a constant
- t is time.

For badly disturbed slopes in ice rich soil, geothermal calculations suggest that a value for α = 0.061 cm/sec$^{1/2}$ is appropriate for the first thaw season. For slopes where a 240 mm peat cover is left intact a value of α = 0.034 cm/sec$^{1/2}$ is predicted.

In the first thaw season the excess pore pressures that can be generated are measured by the parameter R (see Morgenstern and Nixon, 1971):

$$R = \frac{\alpha}{2\sqrt{c_v}}$$

where: c_v = 2.5 x 10^{-3} cm^2/sec for ice rich clay soils.

For the disturbed case R = 0.61 and for the undisturbed case R = 0.34. It is anticipated that construction practice will be such that thermal disturbance is minimized but locally some peat cover may be lost. In the immediate vicinity of the ditch it is clear that disturbance will be caused and locally the organic cover will be lost. For these reasons an average R value of 0.47 has been selected for the first season thaw in ice rich clay slopes. In subsequent years, as the thaw surface penetrates deeper the excess pore pressures are not governed by the simple approach discussed above. Because the rate of thaw becomes less with time the equivalent R value reduces.

5.3.10.3 Stability Analyses: Thawing Slopes

Non-Insulated

These analyses must take into account the following effects:

- The varying thaw bulb dimensions as thaw proceeds deeper into a slope.
- The varying influence of pore pressure effects as thaw proceeds deeper but at the same time the rate of thaw becomes less.
- The changing influence of shear strength components as the effective stresses on the thaw front change.

As thawing proceeds into a slope with a limited width of cleared right of way, the factor of safety is initially governed by the infinite slope equation, illustrated on Figure 5.24. As thaw proceeds deeper, two dimensional effects (shear along the sides of the thaw plug), increase the resistance to failure. This is represented in the analysis in the form of the ratio of the depth of thaw, d, to the right of way width, S. In these equations the (d/S) ratio has been taken as 0.8 d/S to account for the more circular form of the thaw plug, as shown on Figure 5.24.

The following input parameters are selected for the ice rich clay in the Angaro - Lenskiy region:

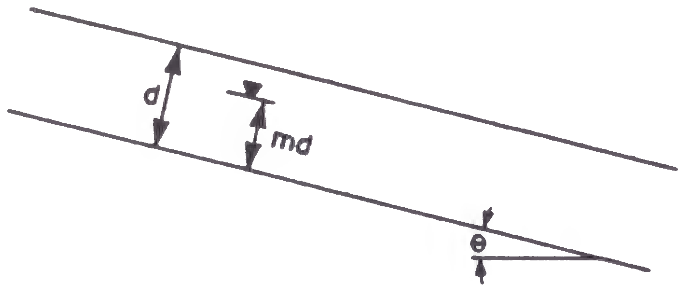
Strength:	c'	= 3.5 kPa, ϕ' = 24.5°
Excess Pore Pressure:	R	= 0.47 Thaw Season 1 and reduced for subsequent years.
Thaw Depth:	From Figure 5.23	
ROW Width:	13 m	
	K_o	= 0.5
	γ	= 1760 kg/m^3, frozen bulk density
	γ'/γ	= 0.5, Thawed Ratio
	a	= 0.12 g Design earthquake acceleration

Calculations then proceed as follows. For a given slope angle, θ, the factor of safety versus depth to thaw front, or shear surface, was obtained as shown for an example of θ = 10° on Figure 5.25.

The factor of safety was first calculated taking into account all effects ($c' + \phi'$), including side shear (ss), for thaw depths ranging from the first thaw season to the maximum depth of thaw for both static and pseudo-static conditions. The theoretical implication of not mobilizing any cohesion, (c' = 0), and no side shear was also analyzed.

A summary plot of the minimum factor of safety versus slope angle is given as Figure 5.26. For all slope angles considered between 8° and 12°, it was found that the first few thaw seasons were critical.

INFINITE SLOPE



NO EXCESS PORE PRESSURES

$$FS = \frac{c'}{\gamma d \sin\theta \cos\theta} + \frac{\left(1 - m\frac{\gamma w}{\gamma}\right)\tan\theta'}{\tan\theta} \quad \text{--------- 1}$$

c', ϕ' = Effective stress strength

d = Depth of thaw

θ = Slope angle

γ = Unit weight of soil

γw = Unit weight of water

md = Phreatic surface

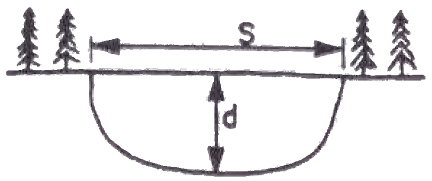
EXCESS PORE PRESSURE

$$FS = \frac{c'}{\gamma d \sin\theta \cos\theta} + \frac{\gamma'}{\gamma}\left(\frac{1}{1+2R^2}\right)\frac{\tan\theta'}{\tan\theta} \quad \text{--------- 2}$$

$\gamma' = \gamma - \gamma w$, $R = \alpha/2\sqrt{2c_v}$

α = A constant $(cm/s^{1/2})$, c_v = Coefficient of consolidation cm^2/s

THAW PLUG



$$FS = \frac{c'}{\gamma d}\left(\sec\theta \ \mathrm{cosec}\,\theta + 2\left(\frac{0.8d}{S}\right)\mathrm{cosec}\,\theta\right) + \frac{\gamma'}{\gamma}\left(\frac{1}{1+2R^2}\right)\frac{\tan\phi'}{\tan\theta}\left(1 + \frac{0.8K_o d}{S}\right) \quad \text{--------- 3}$$

K_o = Earth pressure coefficient

S = Width of thaw plug

0.8 = A factor to account for the non-rectangular thaw bulb

Figure 5.24: Thaw stability analysis

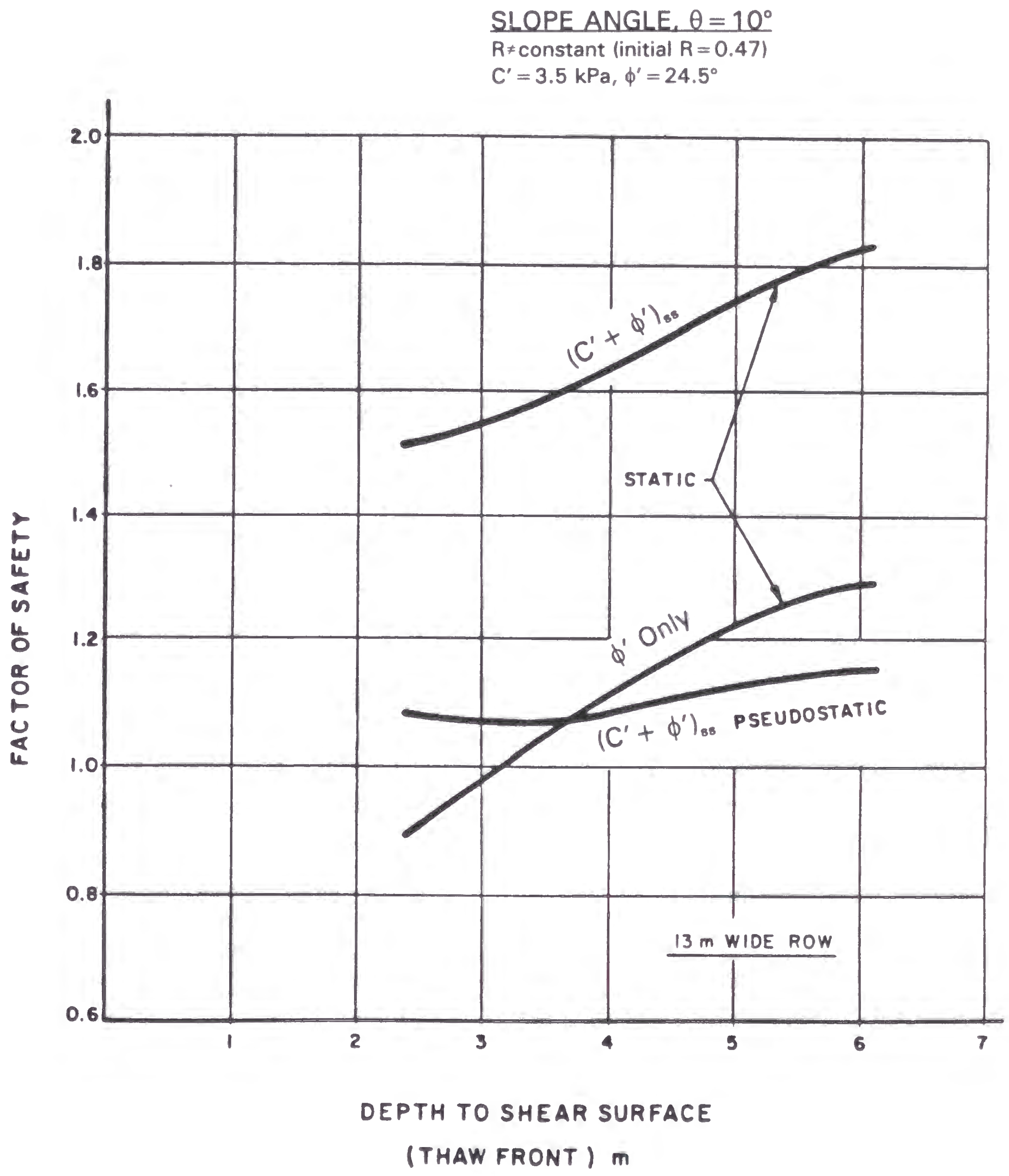


Figure 5.25: Ice-rich clay safety factor versus depth of thaw

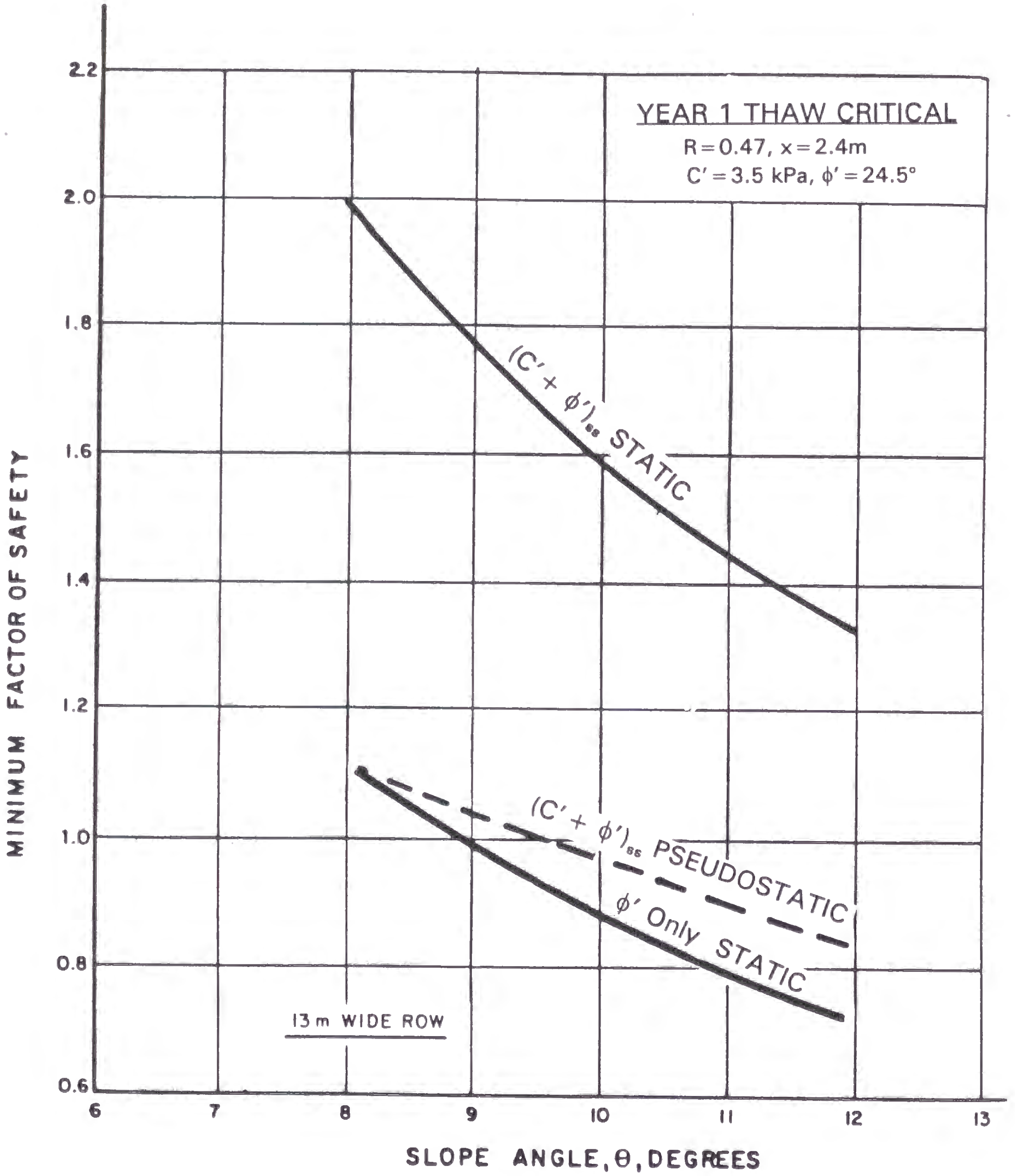


Figure 5.26: Factor of safety versus slope angle

Insulated Surface

Placing wood chips on the slope surface provides a stabilizing influence primarily due to the dramatic reductions in thaw depth. Other options include the use of polystyrene insulation boards with a gravel cover. For wood chips the bulk density γ = 520 kg/m^3, the thermal conductivity ratio k_2/k_1 = 6.65.

The long term case represents the critical condition for insulated surfaces as shown on Figure 5.27. The minimum factor of safety was calculated for four thicknesses of wood chips and different slope angles. The results are presented on Figure 5.28. The point where the pseudo-static factor of safety becomes unity is also shown.

5.3.10.4 Seismic Movement Effects

The pseudo-static method of analysis recognizes the existence of the 'design critical earthquake' by applying a horizontal force to a potential failure surface. This effective horizontal force is the product of the weight of the sliding mass and the fraction, a, of the design horizontal acceleration. This value, a, is the sustained acceleration as normally published in design codes for seismic regions. The factor of safety under this additional effective force has been calculated and is shown on Figures 5.26 and 5.28.

For a certain combination of soil strength properties, representative failure mechanisms and a given slope angle, the pseudo-static factor of safety is reduced to unity under the action of a fraction, N, of the horizontal acceleration. If the design critical acceleration, a, is greater than N, the slope will move. The desirable approach is to keep the pseudo-static factor of safety at or above unity.

If the pseudo-static factor of safety does fall below unity the likely movement can be predicted. Displacements have been calculated and the results for the case of no surface mitigation are shown as displacement versus slope angle in Figure 5.29 (includes analyses for other typical soil conditions). It can be seen that displacements are very small for the first few degrees of slope angle over the angle at which the pseudo-static safety factor is unity. At greater slope angles, the displacements suddenly increase.

The results shown on Figure 5.29 have been presented in a different form on Figure 5.30 The factor of safety versus displacement relationship shows that for several soil and insulation cases, the pseudo-static safety factor can fall to as low as 0.8 before displacements become really significant.

It has been suggested by Newmark (1974) that dynamic slope movements up to 60 cm could be tolerated for a buried pipeline. It is considered that such slope movements may cause plastic deformation but not cracking of a buried pipe. Neither this method nor Newmark's criterion should be relied on in any direct fashion and it is recommended that every effort be taken to ensure a dynamic factor of safety of at least unity. The calculations however demonstrate the reserve capacity that does exist even at the possible condition of pseudo-static failure.

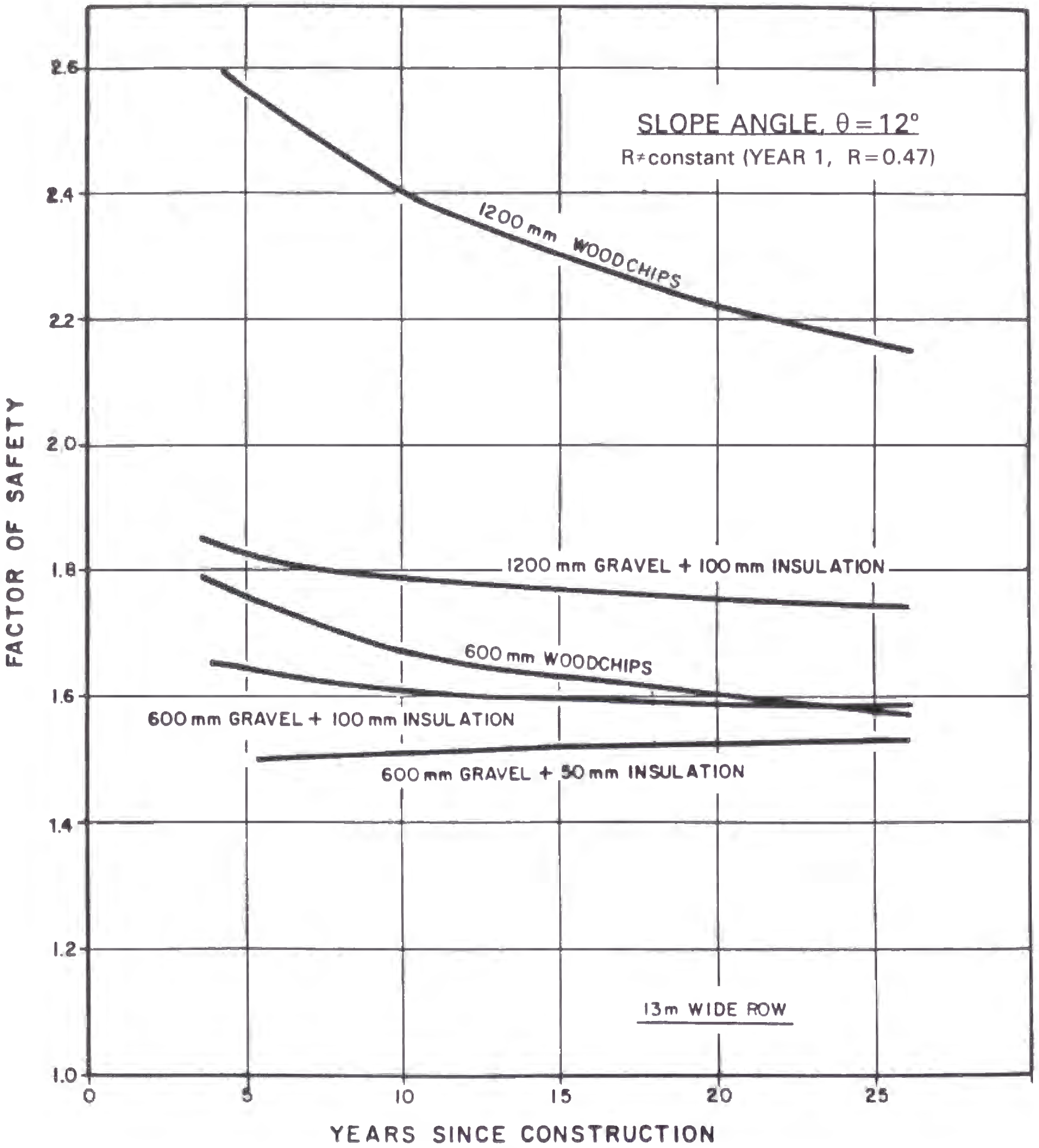


Figure 5.27: Surcharge mitigation - factor of safety versus time (ice-rich clay)

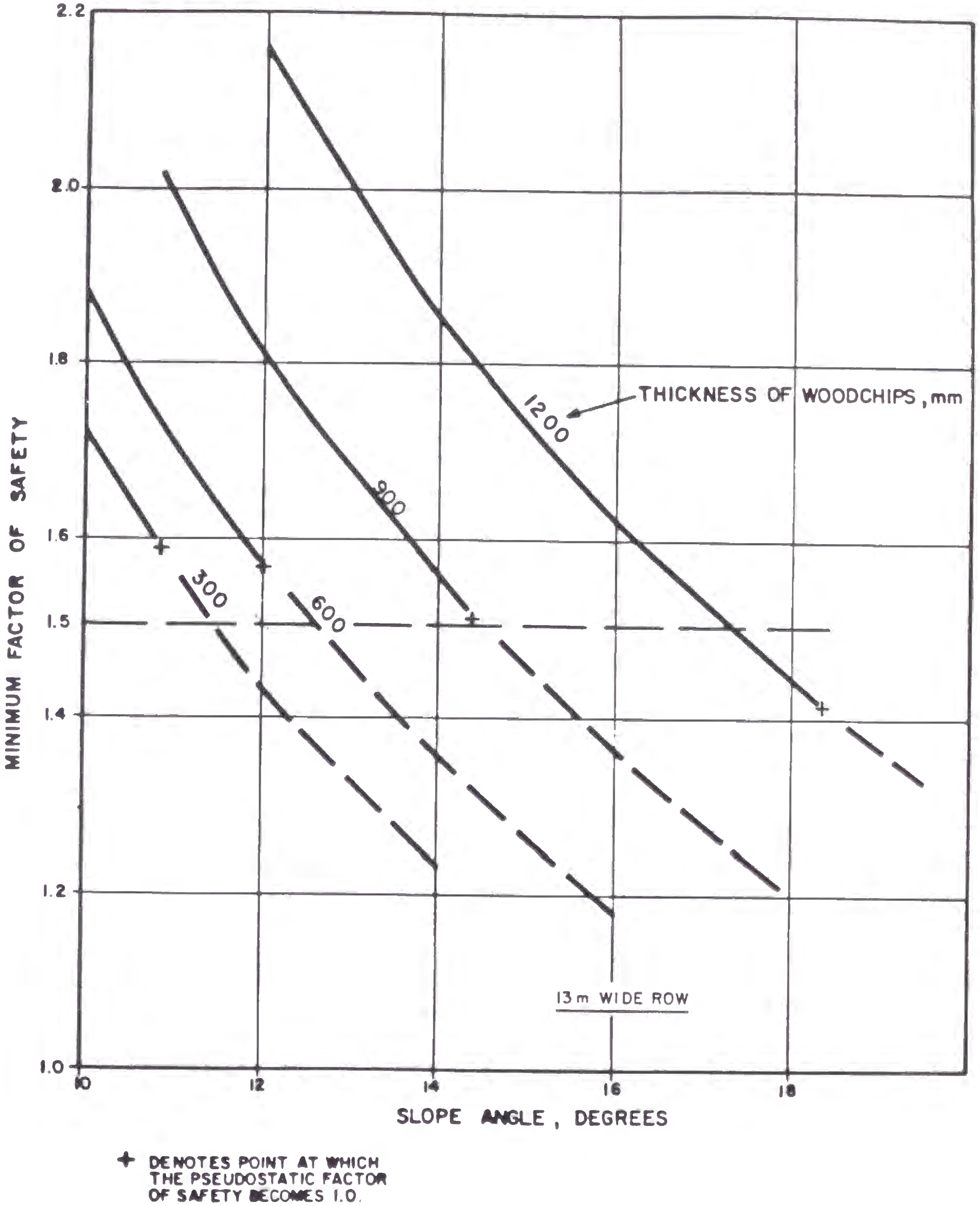


Figure 5.28: Wood chip mitigation-factor of safety versus slope angle (ice-rich clay)

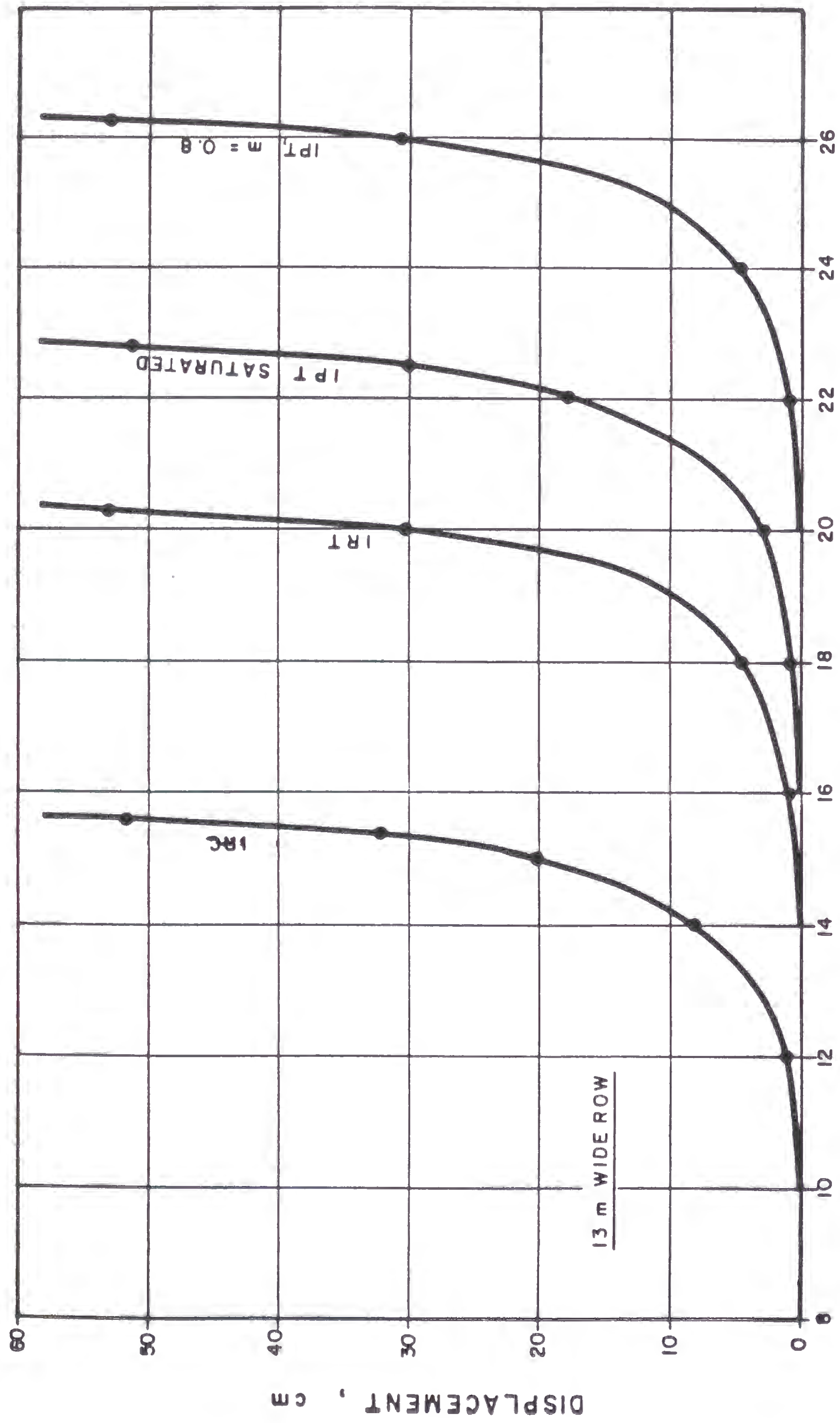


Figure 5.29: Seismic displacements - basic slope design

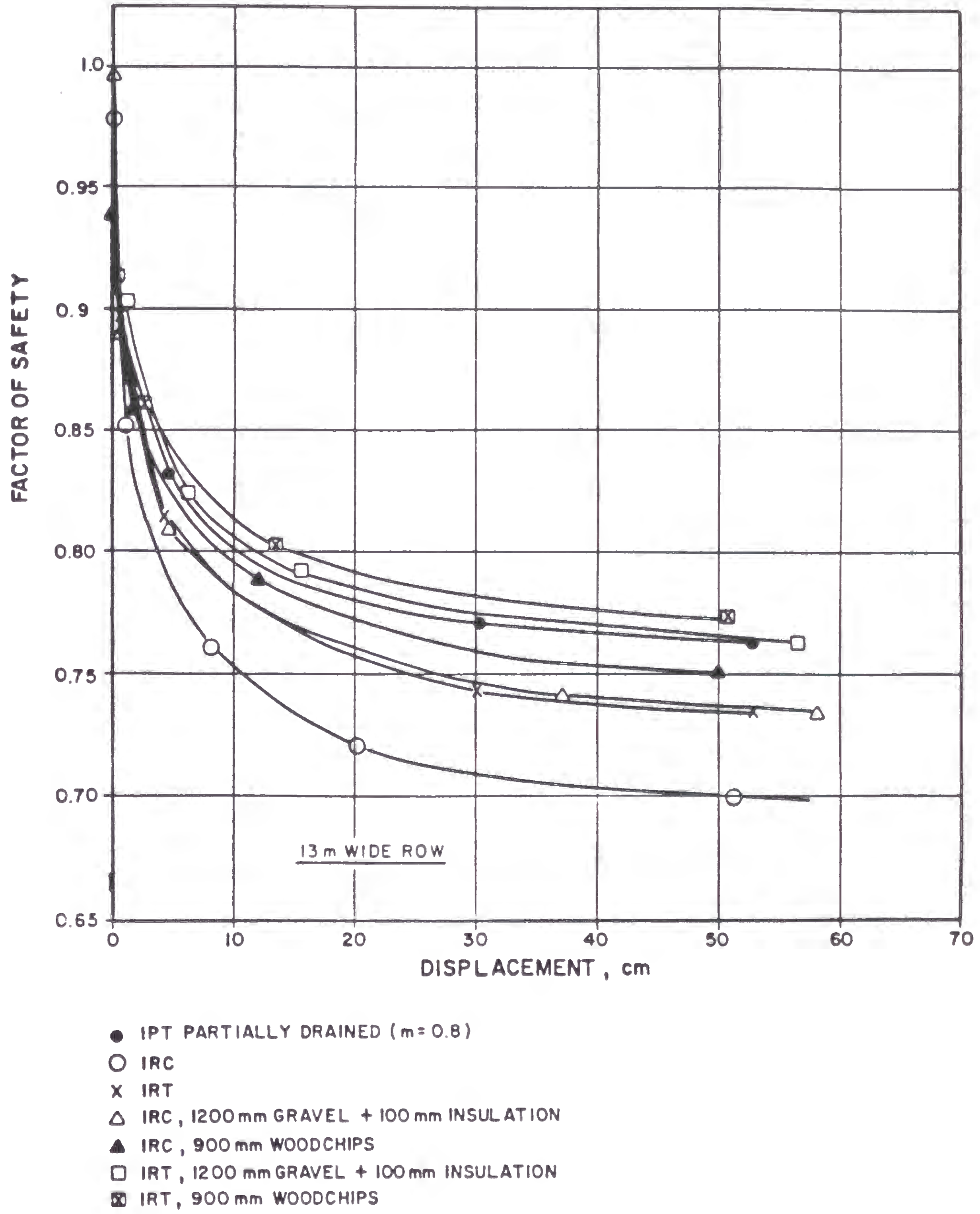


Figure 5.30: Seismic analysis - safety factor versus displacement

5.3.10.5 Effect of Cleared Right of Way Width

As indicated previously the stability analysis for the thawing right of way considers the side shear contribution in the form of the ratio of the thaw depth to width of right of way, d/S. Clearly, the benefit of the side shear decreases as the width of right of way increases.

A series of calculations has been carried out similar to those presented in the previous sections with the value for right of way width, S, changed from 13 to 20, 25 and 30 m. The results with no mitigative surcharge, are shown on Figure 5.31. The basic stable angle is reduced by as much as 2° for each of the typical soils.

Other calculations were carried out to determine the effect of width of the right of way for the wood chip insulation design. The results are shown on Figure 5.32. The effect is similarly to reduce the allowable slope angle for a given thickness of wood chips. It is interesting to note the lesser effect for the thicker wood chips as the very limited thaw depth, d, results in a limited side shear contribution.

5.3.10.6 Backfill Stability

The stability of native spoil as backfill in the trench has been studied. It has been assumed that on slopes, the ditch will likely be excavated by backhoe rather than wheel ditcher. The spoil from backhoe excavation will consist of considerably larger and more irregular sized, frozen chunks than ditcher spoil. The backhoe spoil is considered essentially "unworkable" in its frozen state without some form of processing such as crushing.

The analyses performed have considered two conditions of spoil: unworkable and workable (ditcher spoil). A granular soil, which would be imported to replace unworkable spoil, was also analysed and was assumed to be workable even in winter construction.

The analysis was based on the friction component only of the infinite slope equation (Figure 5.24). The effective cohesion will have been essentially destroyed by the excavation. Because of the porous nature of the loosened backfill material it is assumed no excess pore pressures will develop on thawing (R = 0), even in the more plastic soils.

The results of the analyses are presented in Figure 5.33, in the form of stable angle versus bulk density of the thawing backfill. Two sets of results are presented. The upper curve presents the case where side-shear has been applied along the side of the ditch representing the stability of the full depth of backfill as a mass. In this case, because of some uncertainty about the ditch configuration, a factor of safety of 1.5 has been used to obtain the stable angle.

The lower curve represents a more likely situation of a potential failure at a shallower depth within the backfill. In this case side-shear would not apply. Because this failure is less affected by the uncertain ditch configuration and also would have negligible impact on the pipe, a safety factor of 1.3 has been applied.

Also shown on Figure 5.32 are the expected minimum backfill densities for three major soil types; ice rich clay, ice rich till and ice poor till representing unworkable spoil conditions.

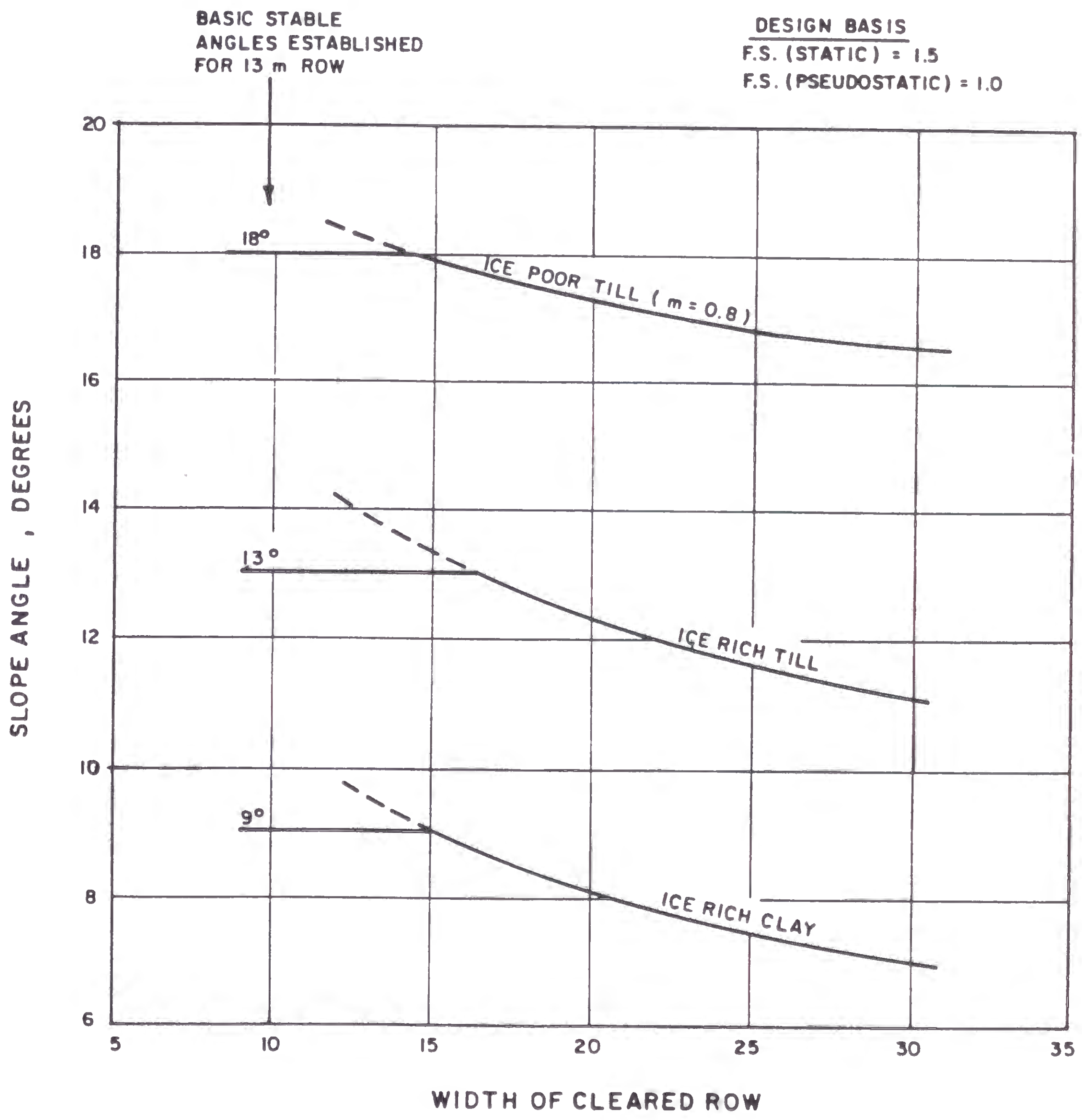
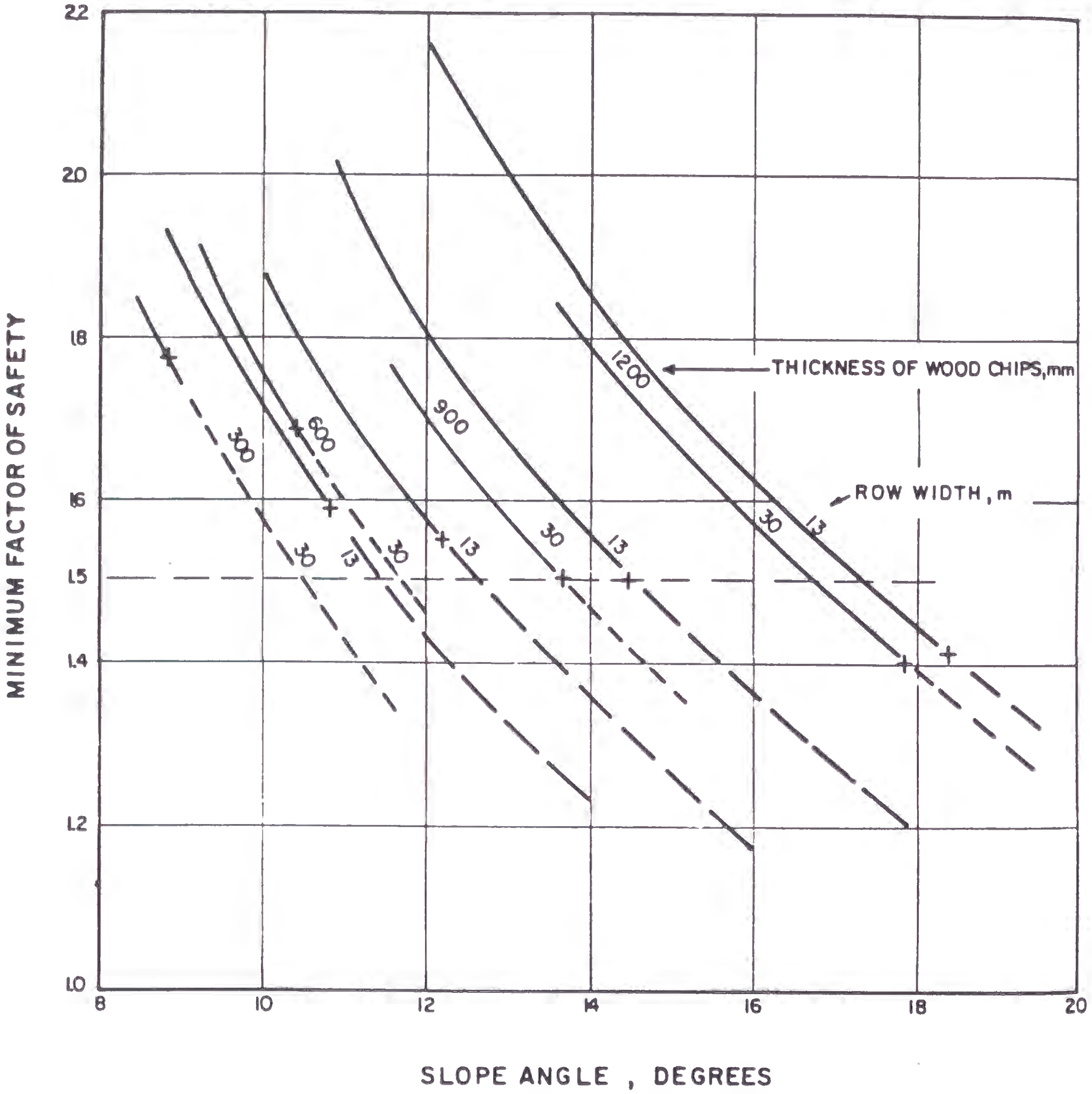


Figure 5.31: Effect of cleared ROW width on basic stable slope angle (no surcharge)



+ DENOTES POINT AT WHICH THE PSEUDOSTATIC FACTOR OF SAFETY BECOMES 1.0

Figure 5.32: Effect of ROW width on wood chip design - factor of safety versus slope angle (ice-rich clay)

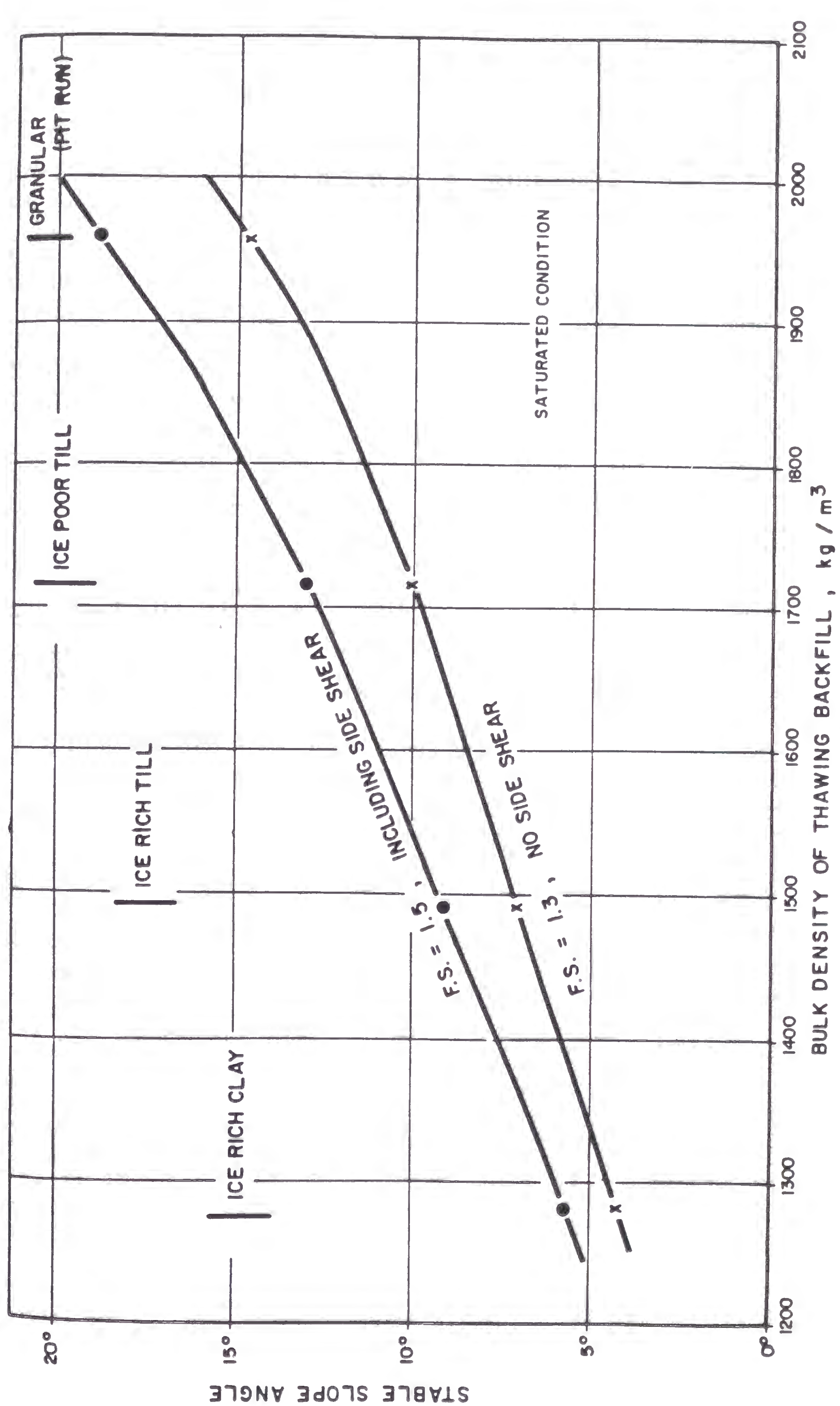


Figure 5.33: Ditch backfill stability chart

5.3.10.7 Recommended Factor of Safety

Factors of safety are set in geotechnical practice having regard to the nature of the geotechnical conditions, the type of structure, the economic implications of failure, and the danger to public safety. Large dams, for example, are usually specified to have a computed safety factor of 1.5. For pipelines in permafrost, the desirable target for static loading conditions, not involving earthquake effects, is to have a safety factor in the range of 1.25 to 1.5. At the same time dynamic/earthquake loading conditions should result in a pseudo-static factor of safety equal to or greater than unity. If such measures are undertaken then the slope and the pipeline will be safe and will easily withstand the ground motion of the design seismic event with the pipeline remaining in safe operating condition.

5.3.10.8 Design Recommendations

For a bare or non-insulated slope surface, 13 m right of way slopes, with every attempt made to minimise surface disturbance, will be stable up to 9°. Slopes greater than these angles will require some form of mitigation.

A summary of the design calculations for wood chips is provided in Figure 5.34. This figure relates the minimum thickness of wood chips versus slope angle in order to keep the static factor of safety greater than 1.5 and the dynamic or pseudo-static safety factor greater than 1.0.

A minimum thickness of 300 mm of wood chips is recommended as shown in Figure 5.34. Where site specific, very high ice content conditions appear to warrant, the maximum thickness of wood chips is specified to essentially prevent thaw over the 25 year period following construction. For these latter conditions the pipe should be above grade for the length of the slope.

For slope angles greater than 18° in ice-rich clay and 20° in ice-rich till, it is recommended that the slope be cut back and insulated.

Wood chips are selected for the following technical reasons:

- Geothermal: wood chips provide a better overall geothermal solution. Gravel tends to promote rather than retard thermal degradation.
- Ease of Construction: Wood chips require less surface preparation than insulation boards. While both gravel and wood chip surcharges require removal of essentially all snow and ice from the slope surface after pipe installation, the insulation board installed with the gravel requires special preparation of a stable, smooth bedding surface.
- Ease of Remedial Action: In the event of local failures to the mitigation measure, wood chips can be readily rehabilitated.
- The known generation of heat during the bio-degradation of the wood chips is considered to be a minor influence in the first year or two following placement.

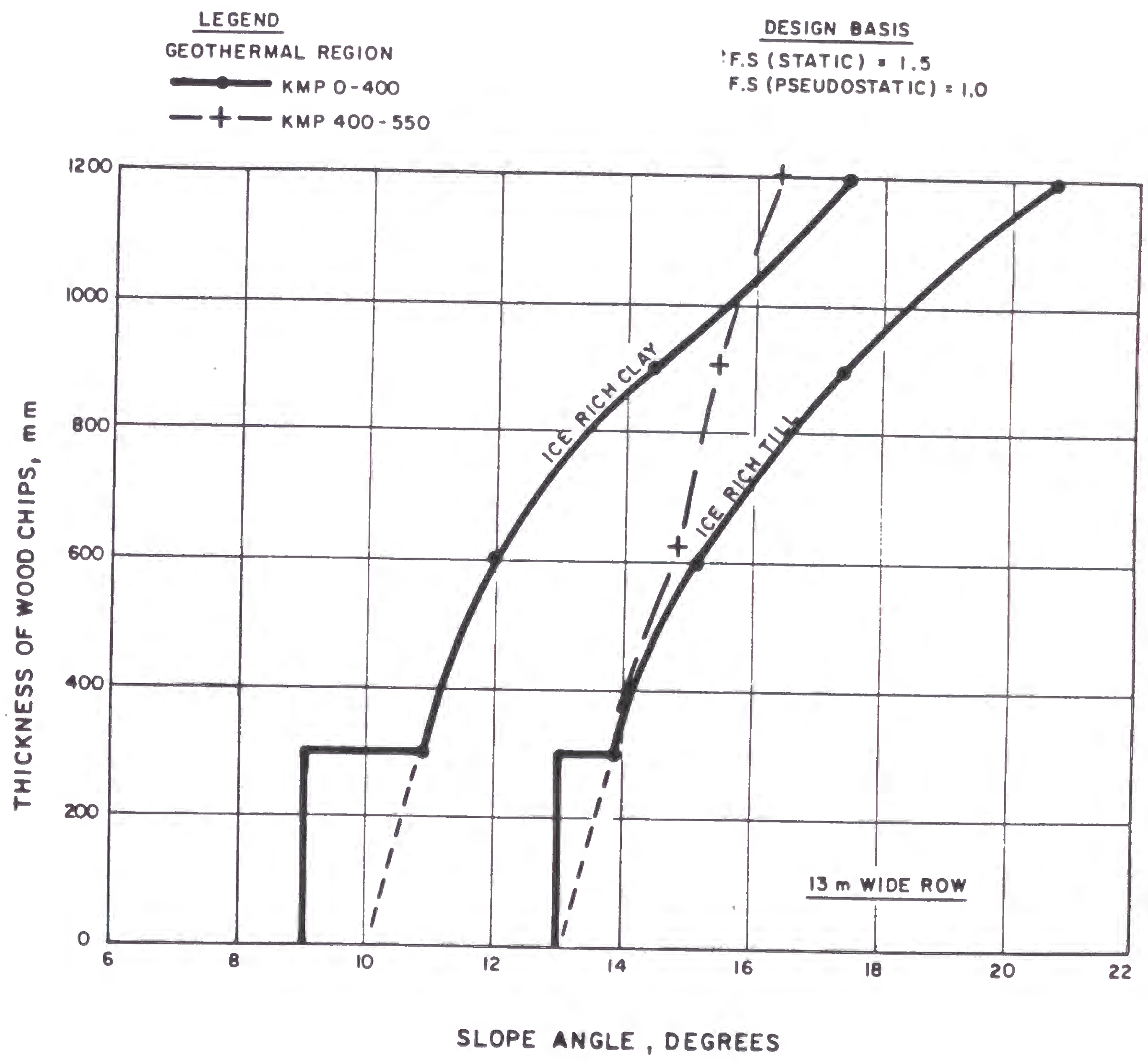


Figure 5.34: Slopes design - wood chip insulation

Native soil can be used as backfill on slopes steeper than 4° and should be replaced or improved on steeper slopes. Improvement may be possible where the native spoil is low in ice content, by means of processing into workable particles of 150 mm nominal size.

5.3.11 Drainage and Erosion Control

The need for drainage and erosion control measures is dependent upon the velocity of the flowing surface water relative to the allowable non-eroding velocity of the flowing surface water relative to the allowable non-eroding velocity for that soil. To prevent erosion, the expected velocities of surface water must be prevented from reaching the erosive velocity for the soil.

The expected flows can be determined using the "Rational" method for the design rainfall event. Knowing that the worst case for erosion will be due to concentration of flows into a defined channel, the calculated flows can be set to "Manning's" equation and the channel area determined. Finally, using the continuity equation, the expected velocity can be calculated and compared with the allowable velocity to assess the potential for erosion and therefore the necessity for drainage and erosion control measures.

Design charts have been developed so that spacings of these structures can be determined. For simplification a single design chart representing average conditions encountered along the pipeline route has been derived, Figure 4.12. This design chart can be used to determine the drainage and erosion control requirements for slopes.

Typically these measures will consist of diversion berms to improve drainage along the slope and restrict surface water flows to non-erosive velocities, and ditch plugs to restrict the movement of ground water through the pipeline ditch by forcing the groundwater within the ditch to the surface where it can be diverted off the right of way. Run off diversions will be used to restrict flows across slopes to non erosive velocities, particularly on cut-slope and insulated slope designs. Figure 5.35 illustrates some of the most common drainage and erosion control measures.

The requirements for drainage and erosion control can be predetermined, however, the exact placement of these requirements should be confirmed in the field during construction to account for post-construction geometry and the intent of the drainage and erosion control mitigation.

5.4 STATIONS AND ANCILLARY FACILITIES

5.4.1 Stations

5.4.1.1 Compression

The compressor stations contain the compressors and prime movers and a large number of other auxiliary systems. A summary of station locations and power requirements is presented in Tables 5.4 and 5.5. A typical layout of a northern, chilled station is shown on Figure 5.36; a southern station is shown on Figure 5.37. Some specific topics that are addressed include:

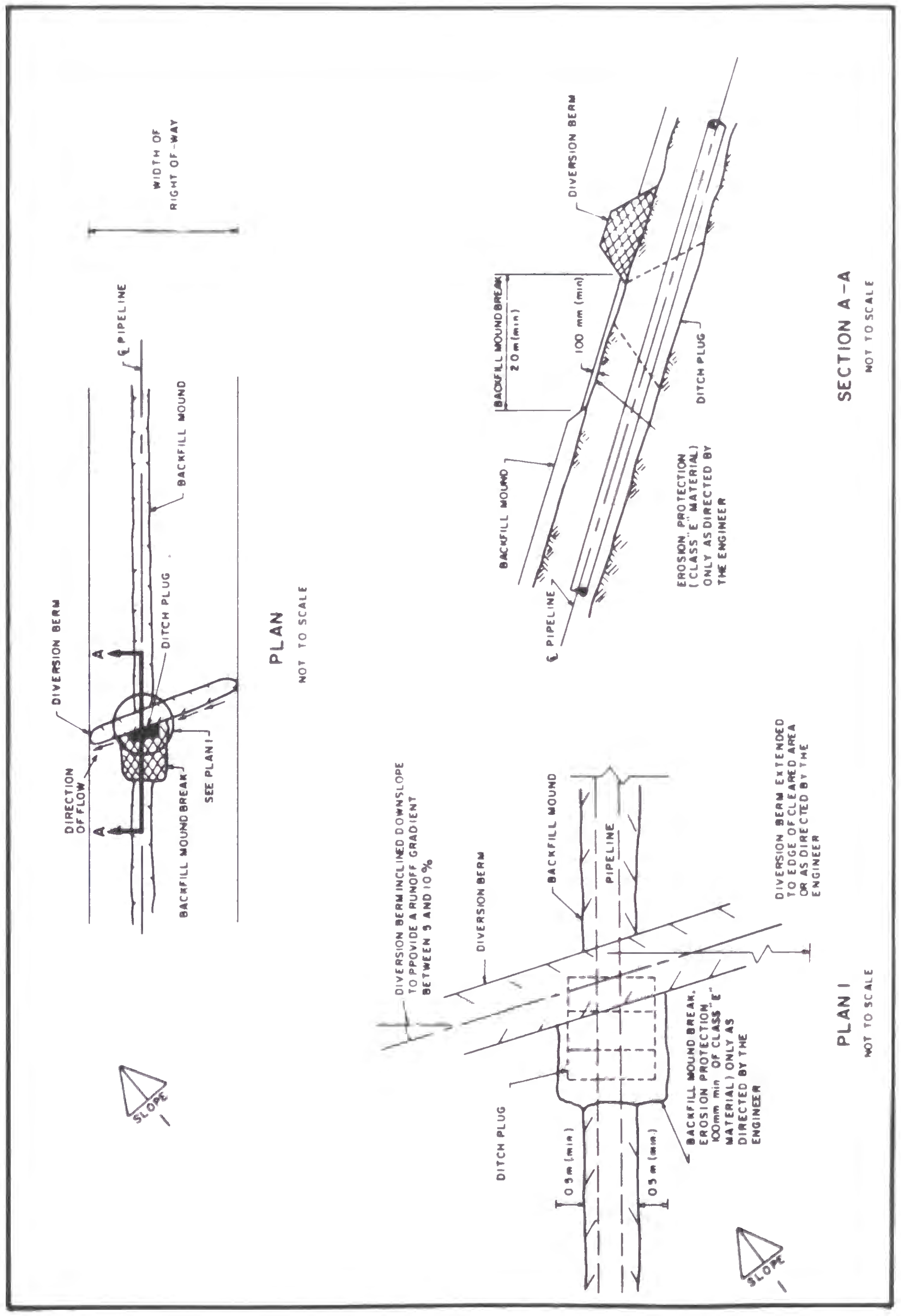


Figure 5.35: Drainage and erosion control measures

1. Station civil design
2. Reliability
3. Compressor unit sizing and selection
4. Chilling and cooling
5. Pig Traps
6. Other systems

The station civil design is concerned with the site layout, site preparation, foundations and supports, buildings, and access. Initially site selection will attempt to choose compressor station sites that are located on fairly stable soil that is less susceptible to thaw settlement. Most sites will be used for stockpiling of material and for location of mainline construction camps. They will be cleared and graded prior to mainline construction and some of the settlement will occur before their conversion from construction sites to compressor stations. A large number of sites will require special fill materials such as gravel, supplemented with board insulation to minimize thaw, where required. Many light buildings will be constructed on-grade with insulation and/or air circulation to minimize heat input to the permafrost. Foundations for compressors and some buildings will use piles, where necessary, to provide long-term stability. Access will be by road wherever possible, however, a heliport would be provided at all sites.

Because a pipeline requires a large number of stations in sequence, there is a significant chance that a compressor unit may not be available for use at some station along the route. Compressor units may be offline for scheduled maintenance or there may be an unscheduled outage. It is generally unacceptable to allow a reduction in throughput during these events since this adversely affects revenue and may violate contractual obligations. To ensure continued capacity to maintain throughput, compressor stations generally install spare compressor units. The most popular patterns are two full-size units (i.e., 2 x 100%) or three units with any two sufficient to maintain throughput (i.e., 3 x 50%). Other strategies, such as using a mobile unit, are also available. For this preliminary design, the 2 x 100% strategy was used at all stations.

No attempt was made to adjust pipeline hydraulics to match the load lines to specific compressor units. This can be an important consideration in detail design since gas turbines are available in only a limited number of discrete sizes. For preliminary design, the Rolls-Royce RB211 gas turbine was considered as a representative model of approximately the right size.

This unit has a nominal (ISO) rating of about 22.8 MW but can deliver much more power (i.e., more than 30 MW) in the winter when air temperatures are low. This higher winter turbine power comes close to the winter design power requirements at most stations. Other turbine models, such as the LM2500+, also come close to matching the power requirements. The exact matching of turbine units to station requirements depends heavily on the annual temperatures at each station and the throughput factor, and is normally undertaken in detail design.

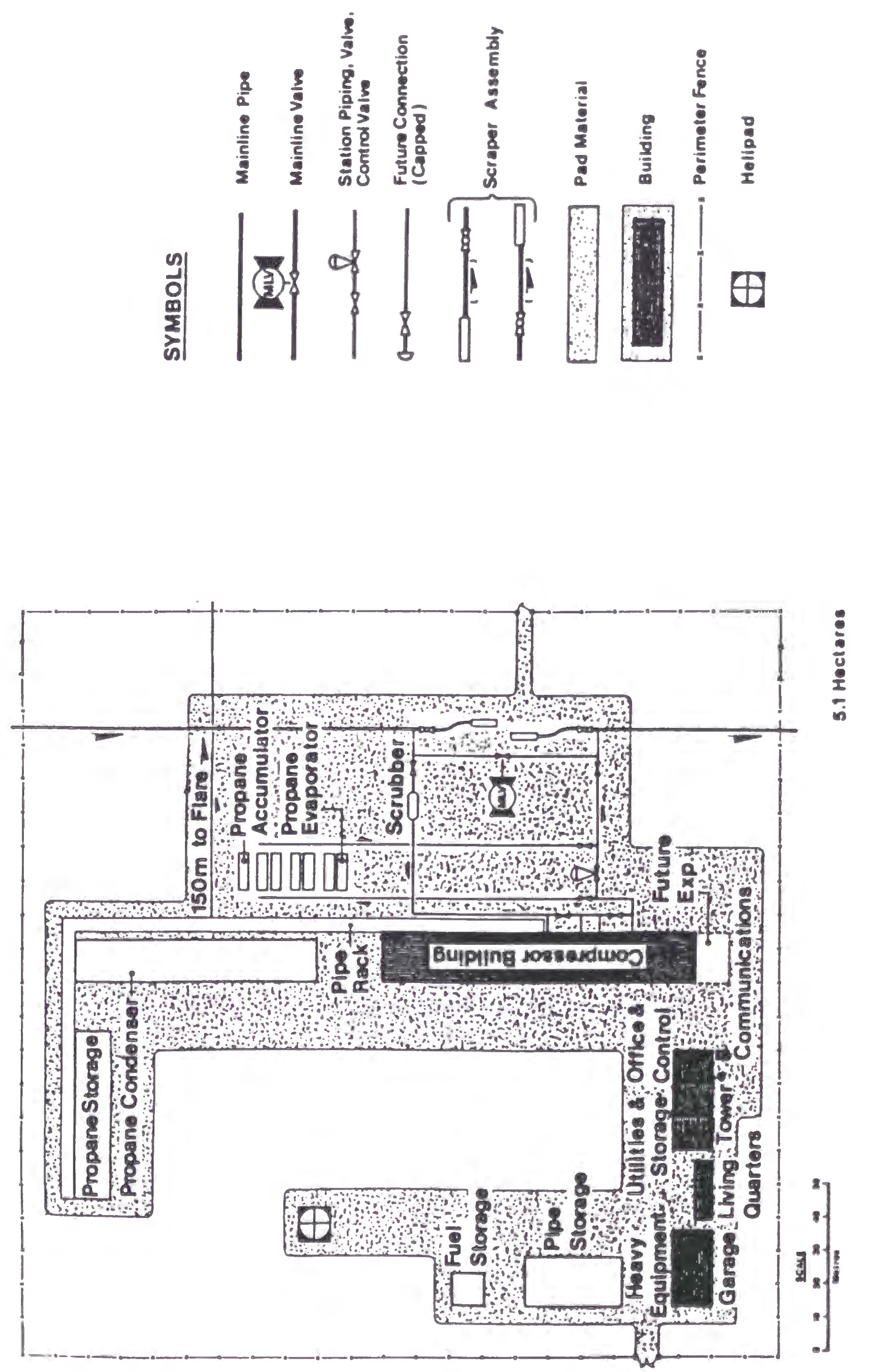


Figure 5.36: Compressor station with gas chilling

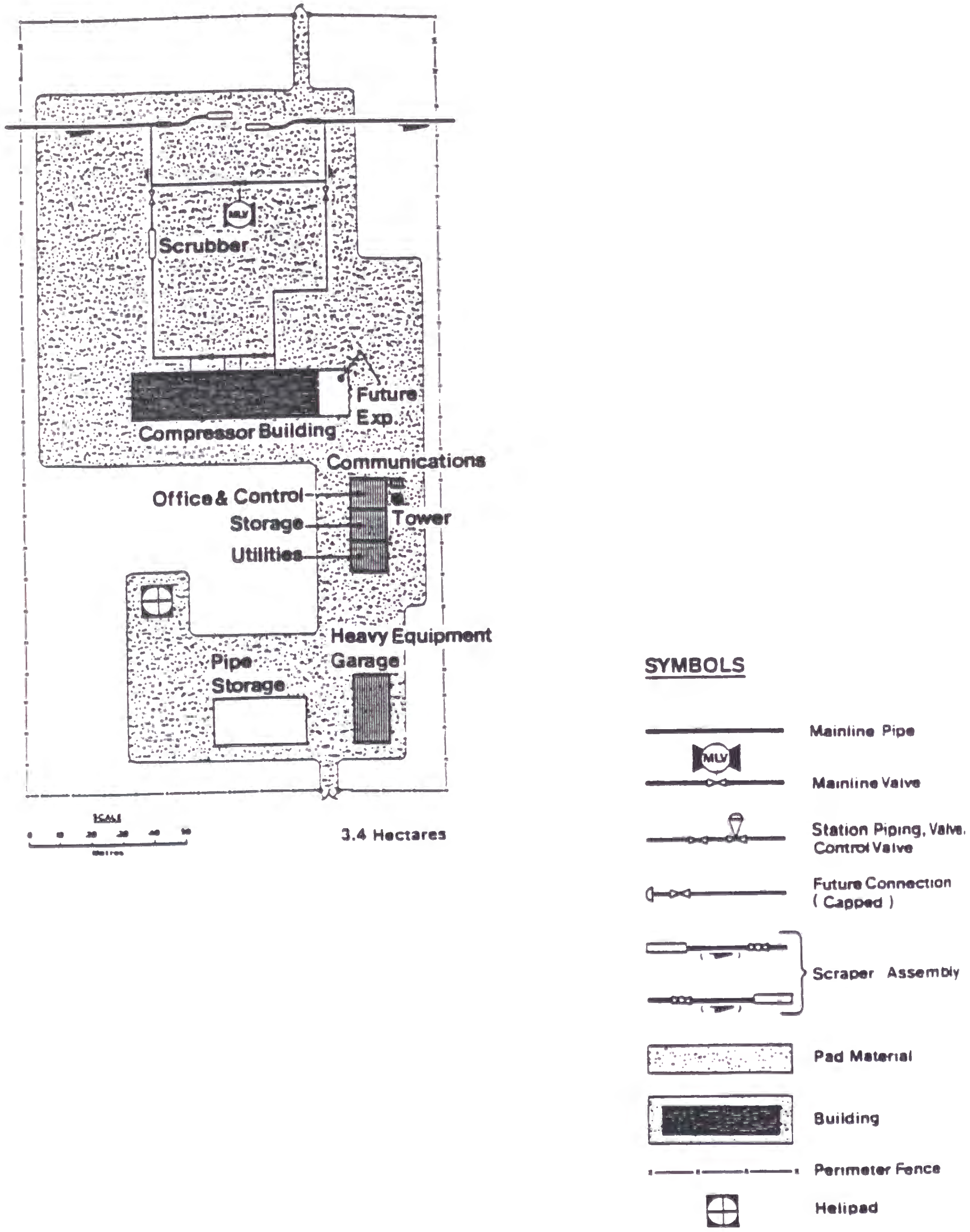


Figure 5.37: Compressor station without gas chilling

TABLE 5.3: Gas Hydraulic Design Simulation Station Summary - Winter Design

KP	Required Power (kW)	Compression Ratio	Flow ($X10^3$) (m^3/d)
0.02	31259.42	1.31	10480.00
114.00	27359.32	1.27	10450.00
254.00	28906.28	1.28	10420.00
376.00	27000.52	1.26	10390.00
550.02	30587.67	1.31	10210.00
662.00	25591.26	1.25	10180.00
844.00	29749.37	1.30	10150.00
996.00	29626.26	1.30	10120.00
1149.00	29603.48	1.30	10090.00
1284.00	26992.76	1.27	10060.00
1409.00	26140.08	1.27	10030.00
1535.00	24880.19	1.26	9850.00
1691.00	24926.15	1.24	9820.00
1838.00	28273.46	1.27	9790.00
1994.00	30023.74	1.28	9760.00
2135.00	32372.61	1.30	9730.00
2273.00	33892.05	1.32	9700.00
2404.00	33940.88	1.32	9670.00
2599.00	33844.36	1.32	9640.00
TOTAL	554,969.85		

TABLE 5.5: **Gas Hydraulic Design Simulation Station Summary - Summer Design**

KP	Required Power (kW)	Compression Ratio	Flow ($X10^3$) (m^3/d)
0.02	25173.17	1.31	8440.00
114.00	14168.13	1.17	8425.00
254.00	13588.88	1.16	8410.00
376.00	13762.11	1.16	8395.00
550.02	9631.89	1.15	8260.00
662.00	13858.24	1.17	8245.00
844.00	8934.82	1.15	8230.00
996.00	14162.48	1.17	8215.00
1149.00	14155.22	1.17	8200.00
1284.00	13934.12	1.17	8185.00
1409.00	14138.13	1.17	8170.00
1535.00	13486.98	1.16	7995.00
1691.00	5223.07	1.13	7980.00
1838.00	10697.91	1.15	7965.00
1994.00	8881.21	1.14	7950.00
2135.00	14439.73	1.17	7935.00
2273.00	15276.22	1.17	7920.00
2404.00	16809.88	1.19	7905.00
2599.00	4978.01	1.12	7890.00
TOTAL	**245,300.20**		

Pig traps will be located at all stations along the pipeline route in order to facilitate cleaning, testing, filling, and especially on-line inspection. A major aspect of pig trap design is minimizing the effects of stresses and displacements. Traditionally this is achieved using anchor blocks. This method would be used for pig traps in conventional terrain; however, it is not suitable for some permafrost soils where anchor blocks may cause, rather than prevent, displacements. The approaches used for permafrost soils will include one or more of the following:

1. Pile supports
2. Expansion loops
3. Anchor struts

The stations will include a large number of subsidiary systems including electric power, building heating and lighting, fire detection and suppression, potable and sewage water, and instrumentation and control. These will largely follow conventional practice with some adjustments for permafrost or remote conditions. The stations will be based on automatic control, either remotely or locally. No operating personnel will be used at stations on a continuous basis. Maintenance personnel will inspect each station, generally on a daily basis. There will be accommodation for maintenance personnel for the intensive maintenance programs that will be carried out on a scheduled annual basis for turbine maintenance and other major chores. This accommodation will also be available throughout the year for emergency crews.

5.4.1.2 Cooling

It is necessary to use chilling at stations in the discontinuous permafrost areas to maintain the temperature of the flowing gas below the maximum of 10°C. Some of the year, this temperature is below the ambient air temperature and so active refrigeration must be used. The exact design of the chilling system is heavily dependent on the throughput factor and the variation of cooling loads and ambient temperatures throughout the year. A simple, representative design has been adopted for preliminary engineering, which has the following features:

1. The chilling system is based on a simple reverse Rankine cycle using propane as the working fluid. There is only a single compression-expansion cycle; no economizer is used.

2. The chilling system compression loads are determined by the summer operating conditions when ambient air temperatures are highest, although the actual chilling loads are lowest in summer because of the effect of the summer throughput factor. The chiller compressors have been selected as two 1 500 kW gas turbines for preliminary design.

3. The evaporator heat exchanger is sized for the winter maximum chilling load. Other vessels are sized proportionately.

4. The aerial coolers for the propane are sized based on the summer condition.

In the conventional terrain in China no cooling is needed until the last four stations. This occurs because the flowing gas temperature steadily increases at each station south of

Ulaan Baatar and the gas temperature eventually exceeds the permitted maximum of 40°C. Because the maximum permissible flowing gas temperature is higher than expected ambient air conditions, aerial cooling is practicable. This is much less expensive than active refrigeration systems for chilling. Some features of the cooling system include:

1. The coolers are only required in winter. This is because the throughput factor reduces energy loads on the system in summer. Because ambient temperatures are lower in winter the coolers will have a high approach temperature and will not be especially large.

2. If the maximum permissible temperature of the flowing gas were increased to 50°C there would be no need for any coolers.

5.4.2 Valve Assemblies

The ASME B31.8 code requires that mainline valves be installed:

1. At stations
2. At the upstream side of the pipeline where it crosses rivers
3. At towns
4. At intervals along the pipeline to segment it:
   - Class 1 - 32 km
   - Class 2 - 24 km
   - Class 3 - 16 km
   - Class 4 - 8 km

The preliminary pipeline designs follow these requirements. In fact it is recommended that block valves be installed on both sides of major river crossings, rather than using only a check valve on the downstream side. Table 5.5 lists all block valves.

The segmentation valves are intended to accomplish the following:

1. Minimize environmental damage and risk to public safety from a break
2. Facilitate maintenance and repair
3. Minimize the gas loss from a break or repair.

The very low population density in some regions of the pipeline makes the risk to the public and environment from a break very low. Additionally, it may be very awkward to reach some valves for regular maintenance and inspection or for emergencies. These valves may not enhance the security of the pipeline and could even detract from it. During detail design, it may be desirable to consider reducing the number of segmentation valves (or eliminating them altogether) in remote Class 1 locations. This would require an exemption by the regulatory authorities.

TABLE 5.5: Compressor Stations and Block Valves

Location	Crossing	K.P.	Comments
Russia			
Meter Station		0.02	Spread 1
Compressor Station No. 1		0.02	3 valves
Block Valve No. 1		28.5	
Block Valve No. 2		57.0	
Block Valve No. 3		85.5	
Compressor Station No. 2		114.02	
Block Valve No. 4		146.0	Spread 2
Block Valve No. 5		178.0	5 valves
Block Valve No. 6	Liga River Crossing	210.0	
Block Valve No. 7	Liga River Crossing	210.5	
Block Valve No. 8		232.0	
Compressor Station No. 3		254.02	
Block Valve No. 9		284.5	Spread 3
Block Valve No. 10		315.0	6 valves
Block Valve No. 11		345.5	
Compressor Station No. 4		376.02	
Block Valve No. 12		406.0	
Block Valve No. 13		436.0	
Block Valve No. 14		466.0	
Block Valve No. 15		496.0	Spread 4
Block Valve No. 16		524.0	7 valves
Block Valve No. 17	Angara River Crossing	540.0	
Block Valve No. 18	Angara River Crossing	540.5	
City of Irkutsk		545.0	
Compressor Station No. 5		550.02	
Block Valve No. 19	Irko River Crossing	565.0	
Block Valve No. 20	Irko River Crossing	566.0	
Block Valve No. 21		598.0	
Block Valve No. 22		630.0	Spread 5
Compressor Station No. 6		662.02	4 valves
Block Valve No. 23		692.0	
Block Valve No. 24		722.0	
Block Valve No. 25		752.0	
Block Valve No. 26		782.0	Spread 6
Block Valve No. 27		812.0	5 valves
Compressor Station No. 7		844.02	
Block Valve No. 28		874.0	
Block Valve No. 29		904.0	
Block Valve No. 30		934.0	
City of Ulan Ude		950.0	Spread 7
Block Valve No. 31		950.0	8 valves
Block Valve No. 32		966.02	
Compressor Station No. 8		996.02	
Block Valve No. 33		1026.0	
Block Valve No. 34		1056.0	
Block Valve No. 35		1086.0	
Block Valve No. 36		1116.0	
Block Valve No. 37	Selenga River Crossing	1140.0	
Block Valve No. 38	Selenga River Crossing	1140.5	
Compressor Station No. 9		1149.02	
Meter Station Russia/Mongolia		1149.02	

Location	Crossing	K.P.	Comments
Mongolia			
Block Valve No. 39		1176.0	Spread 8
Block Valve No. 40		1203.0	5 valves
Block Valve No. 41		1230.0	
Block Valve No. 42		1257.0	
Compressor Station No. 10		1284.02	
Block Valve No. 43		1315.0	
Block Valve No. 44		1346.0	Spread 9
Block Valve No. 45		1377.0	6 valves
Compressor Station No. 11		1409.02	
Block Valve No. 46		1432.0	
Block Valve No. 47		1454.0	
City of Ulaan Baatar		1465.0	
Block Valve No. 48	Gol River Crossing	1470.0	
Block Valve No. 49	Gol River Crossing	1472.0	
Block Valve No. 50		1503.0	Spread 10
Compressor Station No. 12		1535.02	5 valves
Block Valve No. 51		1566.0	
Block Valve No. 52		1597.0	
Block Valve No. 53		1628.0	
Block Valve No. 54		1659.0	
Compressor Station No. 13		1691.02	
Block Valve No. 55		1720.0	Spread 11
Block Valve No. 56		1749.0	5 valves
Block Valve No. 57		1778.0	
Block Valve No. 58		1808.0	
Compressor Station No. 14		1838.02	
Block Valve No. 59		1869.0	
Block Valve No. 60		1900.0	Spread 12
Block Valve No. 61		1931.0	6 valves
Block Valve No. 62		1962.0	
Compressor Station No. 15		1994.02	
Block Valve No. 63		2023.0	
Block Valve No. 64		2052.0	
Block Valve No. 65		2081.0	
Meter Station Mongolia/China		2110.0	

Location	Crossing	K.P.	Comments
China			
Compressor Station No. 16		2135.02	Spread 13
Block Valve No. 66		2163.0	4 valves
Block Valve No. 67		2191.0	
Block Valve No. 68		2215.0	
Block Valve No. 69		2247.0	
Compressor Station No. 17		2273.02	Spread 14
Block Valve No. 70		2299.0	4 valves
Block Valve No. 71		2325.0	
Block Valve No. 72		2351.0	
Block Valve No. 73		2377.0	
Compressor Station No. 18		2404.02	Spread 15
Block Valve No. 74		2436.0	6 valves
Block Valve No. 75		2468.0	
Block Valve No. 76		2500.0	
Block Valve No. 77		2516.0	
City of Chang-Chia-K'ou		2520.0	
Block Valve No. 78		2532.0	
Block Valve No. 79		2548.0	
City of Hsuan-Hua		2550.0	
Block Valve No. 80		2564.0	Spread 16
Block Valve No. 81		2581.0	4 valves
Compressor Station No. 19		2599.02	
Block Valve No. 82	Lake Kuan T'ing	2610.0	
Block Valve No. 83	Crossing	2613.0	
Block Valve No. 84		2641.0	Spread 17
Block Valve No. 85		2668.0	7 valves
Block Valve No. 86		2684.0	
City of Beijing		2700.0	
Block Valve No. 87		2700.0	
Block Valve No. 88		2716.0	
Block Valve No. 89		2732.0	
Block Valve No. 90		2759.0	
Block Valve No. 91		2786.0	Spread 18
Block Valve No. 92		2814.0	5 valves
City of Tianjin		2830.0	
Block Valve No. 93		2830.0	
Block Valve No. 94		2846.0	
Block Valve No. 95		2860.0	
Meter Station at terminal		2825.0	

NOTE: Number of valves does not include block valves at compressor stations and meter stations.

5.4.3 Metering Stations

Metering stations will be installed at each major flow input and output point as well as at national boundaries and at delivery points en-route. Large custody transfer metering stations will use multiple measurement runs with turbine meters. Major stations will also contain other online equipment and facilities to monitor other gas parameters, such as water and hydrocarbon dew points, calorific value, and presence of impurities. This will permit measurement on a mass, volumetric or calorific basis. The large stations will contain their own laboratory facilities unless arrangements can be made with suppliers in nearby towns. The metering stations will also contain calibration facilities and will be open for inspection and audit by the contracting parties as well as government and regulatory authorities.

5.4.4 Operating and Maintenance Facilities

The operation and maintenance of the pipeline system will require a number of field facilities along the pipeline route. The principal types of facilities that will serve different operational functions include:

1. Head office facilities for each national section of the pipeline system
2. Gas control centre
3. Field offices within each country
4. Local O&M facilities at compressor stations

Field facilities may be located in or near communities along the route as well as directly on the right of way. The head office facilities will support general administrative and financial functions, including marketing, gas scheduling, billing, reporting, and related activities. The main gas control centre will provide integrated control of the entire pipeline.

Field offices will consist of a two-tiered arrangement in each country:

1. Division headquarters
2. District facilities

Division headquarters will provide overall guidance and supervision of the district facilities. They will also be responsible for overall support functions such as air transport, and vendor liaison. Each division headquarters will also have two or more district facilities associated with it. District facilities will be equipped to provide all normal, operation and maintenance support of the pipeline and its facilities.

At the lowest level, an inventory of spares, supplies, repair equipment, and emergency response equipment will be maintained at each compressor station along the route.

5.4.5 Communications and Control

5.4.5.1 <u>Communications</u>

Communications will be required for two phases of the project:

1. Construction
2. Operation

During both phases there will be a need for voice communications as well as data communications, such as fax and digital data. The amount of data communications will be significant after the beginning of operation since the control system will depend on long distance communication between points on the route such as stations.

For constructing the pipeline, communications will be needed for administrative, logistical, personnel management, and other construction uses. Much of the communication traffic will be generated by work crews within the construction zones, in camps and in the various offices. Crews include survey, clearing, material staging, facility construction, and pipeline construction. Offices include those of the pipeline company, construction contractors, suppliers and shippers.

For operating the pipeline communications will be needed for administrative, logistical, personnel management, and especially for SCADA data transfer. Communication traffic will be generated by crews and vehicle operators along the route, staff at the control centres, compressor stations, meter stations, and various other facilities and offices. SCADA data traffic will be exchanged from the control centre to all the sites equipped for telemetering data, such as compressor sites.

There are several major categories of communication system that can be used for construction and operation:

1. Local communications infrastructure such as those offered by the local telephone companies and other communications companies.

2. Microwave facilities constructed for the project

3. Satellite communications facilities with ground facilities constructed for the project

4. Fibre optic cable system constructed for the project

It is intended to use the existing communications infrastructure as much as possible during both construction and operation. However, for the purposes of this study, it has been assumed that the communications requirements during construction and operation will be provided by leasing access to existing satellite systems (with a backup system).

5.4.5.2 SCADA

It is intended that the pipeline system will be monitored and controlled using a supervisory control and data acquisition (SCADA) system. The system will use computers at the control centre to communicate with remote terminal units (RTUs) at compressor stations, metering stations, O&M centres, mainline valves and other sites along the pipeline. The operators at the control centre will be able to initiate, control and monitor all operations from the control centre.

5.4.6 Cathodic Protection

The cathodic protection system will be one component of the provisions to ensure the integrity of the pipeline and the prevention of corrosion. Other components include:

1. Exterior (and possibly interior) pipeline coatings,
2. Quality control that will be exercised over the flowing gas at the input meter stations.
3. Periodic monitoring of the pipeline condition, primarily with smart pigs.

The cathodic protection system will consist of impressed current facilities and would be located at sites such as compressor stations where power is readily available. Anode beds may be used to supplement the impressed current facilities. The type of ground bed that will be used depends on soil resistivity tests including the effects of permafrost. Shallow anode beds should generally be sufficient in discontinuous permafrost zones; deep anode beds would be considered for soils of very high resistivity. The pipeline will have test leads attached at regular intervals. The test leads will be brought to above ground terminal posts to facilitate measurement of resistivities, potentials, and other cathodic protection parameters.

The exact details of currents, rectifier beds, ground bed and anode bed designs will be completed after a resistivity survey is conducted by a specialist contractor shortly after the completion of each pipeline section.

5.4.7 Foundations for Facilities

5.4.7.1 General

All heated structures and hot pipes should be elevated above the ground by 0.6 to 1.0 m to isolated the heat from the facility from the ground. For structures with heavy floor loads, such as warehouses and garages, it is expected the floor would be constructed on-grade. In this case, some form of heat interception must be employed. The foundation designs must assume ice-rich permafrost, which is nominally -1°C. Because, the permafrost is very close to the thaw point, the foundations will require special measures to maintain the ground frozen.

A gravel pad should be placed over the entire area of any compressor station or other ancillary facilities on permafrost. The minimum thickness should be 500 mm on stable ground and 1000 mm on ice rich permafrost. There will be a gradual thawing of the underlying permafrost which could result in thaw settlement of the pad. There will be a

requirement to maintain the surface, especially in the first few years following construction. The surface of the pad should be graded to maintain good drainage towards the edges. In areas where vehicles will travel, the surface should be completed with a good quality road gravel.

5.4.7.2 Pile Foundations

Pile foundations will be suitable for all heated structures that can be elevated off the surface of the gravel pad and for non-heated structures. Since the effect of the gravel pad will be to warm the permafrost, it will be necessary to use thermal piles for all pile foundations. These piles are designed on the basis of the adfreeze between the pile and the surrounding ground. Based on the thermal piles, the design adfreeze bond can be taken as 60 kPa. The load capacity for a range of pile diameters and depths have been calculated and are shown on Table 5.6.

Table 5.6 Range of Thermo Pile Capacities (assuming -1.5°C to -3°C)

	PILE CAPACITIES , kN		
PILE DIAMETER	100 mm	200 mm	300 mm
Depth, m			
7	80 - 100	130 - 170	170 - 220
10	120 - 170	210 - 270	270 - 350
12	160 - 210	260 - 340	340 - 440

5.4.7.3 Foundations for On-Grade Buildings

The recommended foundation system for heated, on-grade buildings is the thermo-siphon design. This will require typically one thermosyphon for every 50 to 70 square metres of floor space, depending on the floor temperature and the specific ground conditions. Alternatively, a system of cooling pipes may be installed beneath the floor slab, with a series of thermosyphons providing the cooling, as was illustrated on Figure 4.11.

5.5 COST ESTIMATION

5.5.1 Estimated Capital Costs

In determining capital expenditures and operating and maintenance costs, it has been assumed that all material will be purchased at competitive international prices; construction productivity will reflect North American rates under similar conditions and labour compensation. Similarly, the operations plan reflects North American practices and costs for labour and equipment.

On the preceding basis, it has been determined that the pipeline system can be constructed for an estimated capital expenditure of $6,883.4 million. (All dollar figures in the study are based on 1996 US dollars.) The capital expenditure breakdown among the countries is as follows:

	US$ millions	Percent
Russia	2,854.7	41
Mongolia	1,916.3	28
China	2,112.4	31
TOTAL	6,883.4	100

For the base case, the total capital cost is $7,701.7 million, capitalizing only the interest on the debt portion during the construction years. For other financial parameters in the sensitivity studies the capital expenditure remains the same but the total capital cost will vary with the changes in financial parameters.

5.5.2 Estimated Operating Costs

The estimated costs for operating equipment are presented in Table 5.7.

Table 5.7 Cost Summary - Other Facilities and Operations and Maintenance Equipment

	Russia	Mongolia	China	Total
	($US millions)			
Meter Stations	19.8	10.1	18.2	48.1

Operations and Maintenance Facilities	Russia	Mongolia	China	Total
	($US millions)			
Head Office	6.5	6.5	9.75	22.75
Division Headquarters	3.6	3.6	6.0	13.20
District Offices and Maintenance Bases	7.35	7.35	5.13	19.83
Office Equipment	1.7	1.7	2.2	5.60
Operations and Maintenance Equipment	48.3	47.9	36.9	133.10
Communications	1.20	1.0	0.8	3.00
TOTAL	68.65	68.05	60.78	197.48

An estimate of the operations and maintenance labour and other significant costs are presented in Table 5.8.

TABLE 5.8 Summary of Annual Operating Costs (Thousands $US)

	Russia	Mongolia	China	Total
Head Office	18,000	18,000	18,000	54,000
Division Offices	1,760	1,760	1,760	5,280
District Offices	19,110	14,910	11,620	45,640
Communications	185	126	139	450
Sub Total	39,055	34,796	31,519	105,370
Fuel	23,450	13,100	10,350	46,900
Sub Total	62,505	47,896	41,869	152,270
Municipal Taxes	31,950	21,400	23,650	77,000
TOTAL	94,455	69,296	65,519	229,270

5.5.3 Cost of Transportation

The average cost of transportation is $\$45.50/10^3m^3$ or $1.29/mcf based on the given financial parameters and on obtaining an internal rate of return of 15% over a 20 year period. The after-tax return to the pipeline system in each country, for the base case and all other cases, would be in proportion to the initial capital expenditure in the country, as noted earlier.

In addition to the base case, a number of sensitivity studies were undertaken to indicate the effect of changing certain financial parameters, the project life and fuel costs. The results of the cost-of-transport analyses are summarized in Table 5.9. They consist of the following analyses:

1. Case 1, using the base-case interest and return-on-equity rates (10% and 15%, respectively) without inflation of operating costs or revenues. This analysis consists of an aggregate cost of transportation table for the complete pipeline system.

   A single flat-rate cost of transportation applicable to all deliveries, irrespective of their distance from the gas field, is called a "postage-stamp" cost of transportation, analogous with the practice of most post offices which charge a flat rate for any destination within a country. A common alternative is a cost of transportation based on the distance the gas is transported as either a "mass times distance" cost of transportation or a "volume times distance" cost of transportation.

TABLE 5.9 Feasibility Study - Cost of Transportation Alternatives

Case No.	Equity Return %	Debt Interest %	Project Life Yrs.	International Rate %	Fuel Cost $/mcf	COST OF TRANSPORTATION $/10^{3}m^3 Year 1	Year 5	Year 10	Year 20	$/mcf Year 1	Year 5	Year 10	Year 20
1	15	10	20	0	1.00	45.5				1.29			
2	15	10	20	2.0	1.00	41.40	44.9	49.5	60.4	1.17	1.27	1.40	1.71
3	13	8	20	0	1.00	39.6				1.12			
4	13	8	20	2.0	1.00	35.9	38.9	42.9	52.3	1.02	1.10	1.21	1.48
5	15	10	20	0	2.00	46.9				1.32			
6	13	8	30	0	1.00	37.4				1.06			
7*	15	10	20	0	1.00	58.3	50.7	41.2	22.2	1.65	1.44	1.17	0.63
8*	13	8	20	0	1.00	51.2	45.0	38.2	21.7	1.45	1.27	1.05	0.61
9*	13	8	30	0	1.00	46.2	42.1	37.0	26.9	1.31	1.19	1.05	0.76

*Based on annual return on equity - all others are for an internal rate of return over project life.

Other structures are possible such as a mixed cost of transportation where costs of transportation are constant within a specific region but vary from one region to another. Still other options are to charge a cost of transportation that depends on the type of customer (e.g., industrial versus domestic), the demand throughput factor, or the gas contract terms (e.g., interruptible service).

The distribution of the cost of transport in time is also a matter to be determined. Again, many alternative methods are available. Two were considered for the analyses presented here:

- a "levelized" cost of transport. This structure uses a single cost of transportation rate over the entire operational life. The cost of transport is sufficient to generate the required return on equity on a discounted basis but does not generate the same return on equity each year.

- an "annual" cost of transport. This structure adjusts the cost of transport annually to generate the same return on equity each year of operation. Note that the rate of annual return is higher than the nominal target rate since cash flows are discounted to the beginning of construction rather than the beginning of operation.

The levelized cost of transport was used for the base case.

2. Case 2, using the base-case interest and return-on-equity rates (10% and 15% respectively) with 2 % inflation of operating costs and revenues. This analysis consists only of an aggregate cost of service table for the complete pipeline system.

3. Case 3, using the base-case interest and return-on-equity rates (10% and 15% respectively) each reduced by 2%, without inflation of operating costs or revenues. This analysis is for the complete pipeline system.

4. Case 4, using the base-case interest and return-on-equity rates (10% and 15% respectively) each reduced by 2%, with 2% inflation of operating costs and revenues, for the complete pipeline system.

5. Case 5, which is the same as Case 1 except that fuel gas cost is increased to $70.60/10^3 m^3$ from $35.30/10^3 m^3$.

6. Case 6, using the base-case interest and return-on-equity rates (10% and 15%, respectively) each reduced by 2% without inflation of operating costs and revenues, but with project life extended to thirty years. This analysis is for the complete pipeline system.

7. Case 7, using the base-case interest and return-on-equity rates (10% and 15%, respectively) the equity return has been capitalized in the construction period. The return on equity has been taken as 15% in each operating year, giving a rate-based cost of transportation for each year.

8. Case 8, using the reduced interest and return-on-equity rates (8% and 13% respectively) as in Case 3. The equity return has been capitalized in the construction period. The return on equity has been taken as 13% on each operating year giving a rate-based cost of transportation for each year.

9. Case 9, the same as Case 8 except that the project life has been extended to 30 years.

It is obvious that most of the input parameters have a significant influence. These analyses indicate that the project is economically viable under several scenarios. This preliminary information will provide assistance in selecting financial parameters for the project in future financial analysis.

5.5.4 Comparison with North American Costs

The capital and operating costs associated with pipelines have historically been of interest to a wide spectrum of people involved in the petroleum industry. This interest is common within various sectors including the producers, transporters, buyers, service companies, and regulatory agencies.

Considerable information is published yearly on the costs, however, there are often wide variations in these costs, which are market driven and may reflect market demand or changes in labour agreements. Pipe and labour costs represent the two major components of the installed capital costs for any pipeline system. The cost of pipe and other materials are influenced by international demand and, therefore, remain reasonably consistent regardless of location. Construction costs are different and are largely controlled by the level of domestic competition.

Construction costs vary considerably due to terrain conditions, the length of any new construction, mobilization costs, existing infrastructure, population densities and other such factors. Both bedrock and swamp conditions have a great impact on costs due to their impact on productivity related to the increase in activities, the degree of difficulty and the additional material requirements.

These construction cost factors are demonstrated on Figure 5.38, comparing mainline investment for two major natural gas transmission companies within North America. The upper line represents the cost data from a company whose system is transcontinental and covers the Precambrian Shield consisting of extensive hard bedrock, interspersed with swamp. The lower line presents the costs of a major company whose system is more regional and has little exposure to rock and difficult terrain conditions. These costs are adjusted to reflect the effects of inflation. A point of interest is the impact following the 1986 collapse of world oil pricing. In more recent years, the upper line shows continuing cost reductions due to increasing percentages of large diameter pipe, while the other shows an increase in costs.

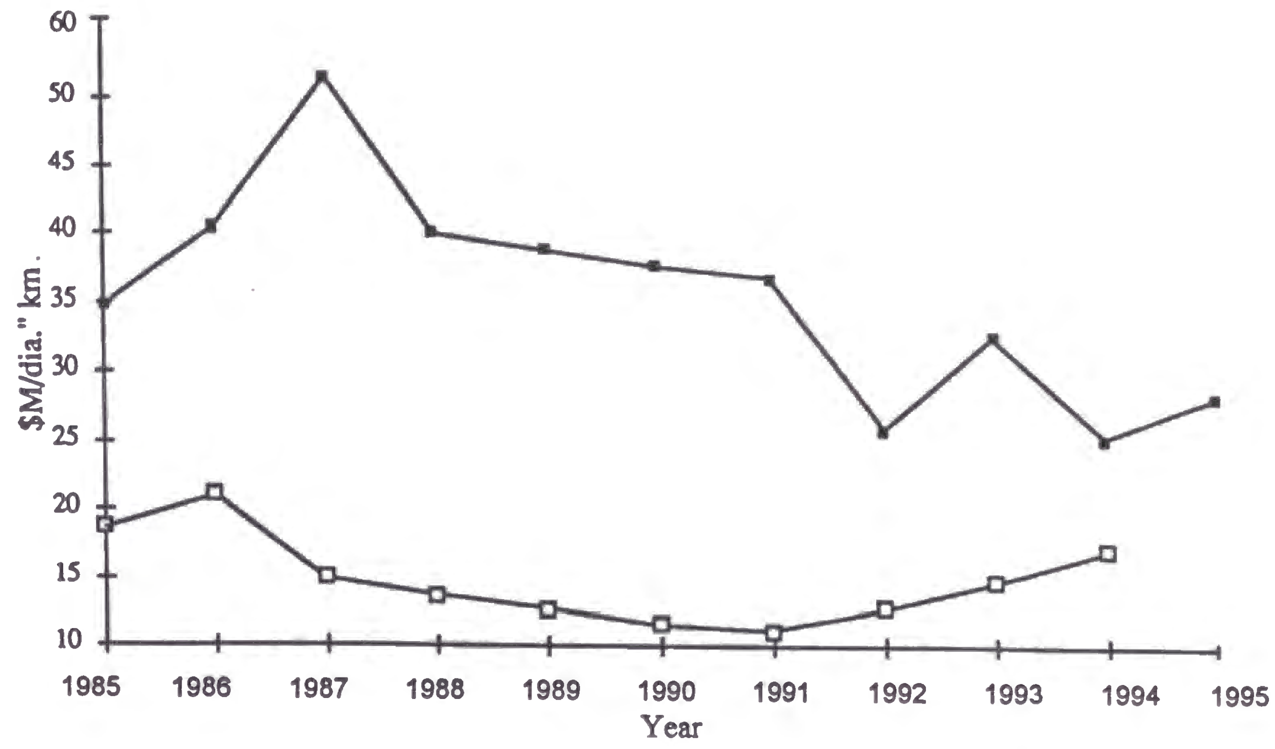


Figure 5.38: Pipeline cost trends comparison - NPS 6 to NPS 48

Historically, the cost variances in the capital costs of pipeline systems varied rather dramatically between countries and regions of the world. As design standards and regulations have evolved to a level of relative sophistication, and international sourcing of materials and services is common, the cost variances are much less pronounced. Utilizing basic rule-of-thumb costs and knowledge of the country, representative pipeline system costs can be generated. This is primarily due to the fact that material costs represent approximately fifty percent or more of the installed pipeline costs, exclusive of facilities. Although these costs vary, current pricing can be determined with relative ease. The other major cost component in construction is labour. The basic lay price can be adjusted to reflect terrain conditions and other local factors. Equipment and associated costs represent approximately the same costs as the labour component and this relationship provides a check in the estimating process. If the equipment/labour relationship changes due to the availability of equipment or for job creation, the impact is not as significant as may be perceived. However, production rates suffer, thus increasing the job duration and impacting on all costs including overheads, financing and lost revenue.

Terrain conditions that effect construction, each have rule-of-thumb values which can be applied to the base lay cost to provide a reasonable total construction cost. The other costs, combined, are not significant as a percentage of total costs. An exception to this could be the applicable interest charges for a major multi-year project. Overall percentage relationships change only slightly and gradually and have only minor impact.

Figures 5.39 and 5.40 illustrate these various components. Figure 5.39 shows actual costs incurred by a major natural gas transmission company. The construction costs in this particular example are higher than normal due to terrain conditions. Other costs include land, engineering, contingencies, overheads and AFUDC. Figure 5.40 is from an August, 1997 Oil & Gas Journal covering North American pipeline costs.

Costing terminology common to the industry includes the following:

Construction	- \$/ diameter inch/ metre (or foot)
	- \$/ diameter inch/ kilometre (or mile)
Mainline costs	- \$/ diameter inch/ kilometre (or mile)
Compression	- \$/ installed MW (or horsepower)

Section 5.5.1 has given the total capital expenditure for construction, including all facilities. The costs for the compressor stations have been deducted (as well as the proportional amounts for engineering and contingency) from the expenditure costs, for the purposes of calculating the unit pipeline construction cost. The resulting unit construction cost for the proposed pipeline system is about US\$ 29 500/ diameter-inch/ kilometre. It should be noted that typical North American unit costs range from \$ 14 000 to \$ 32 000/ diameter-inch/ kilometre, for major regulated transmission pipelines. The higher costs represent very difficult terrain conditions and the lower costs represent easy prairie conditions. The estimated construction costs for the ESFE project are intentionally around the high end of the North American range, given the permafrost, rock and congested conditions.

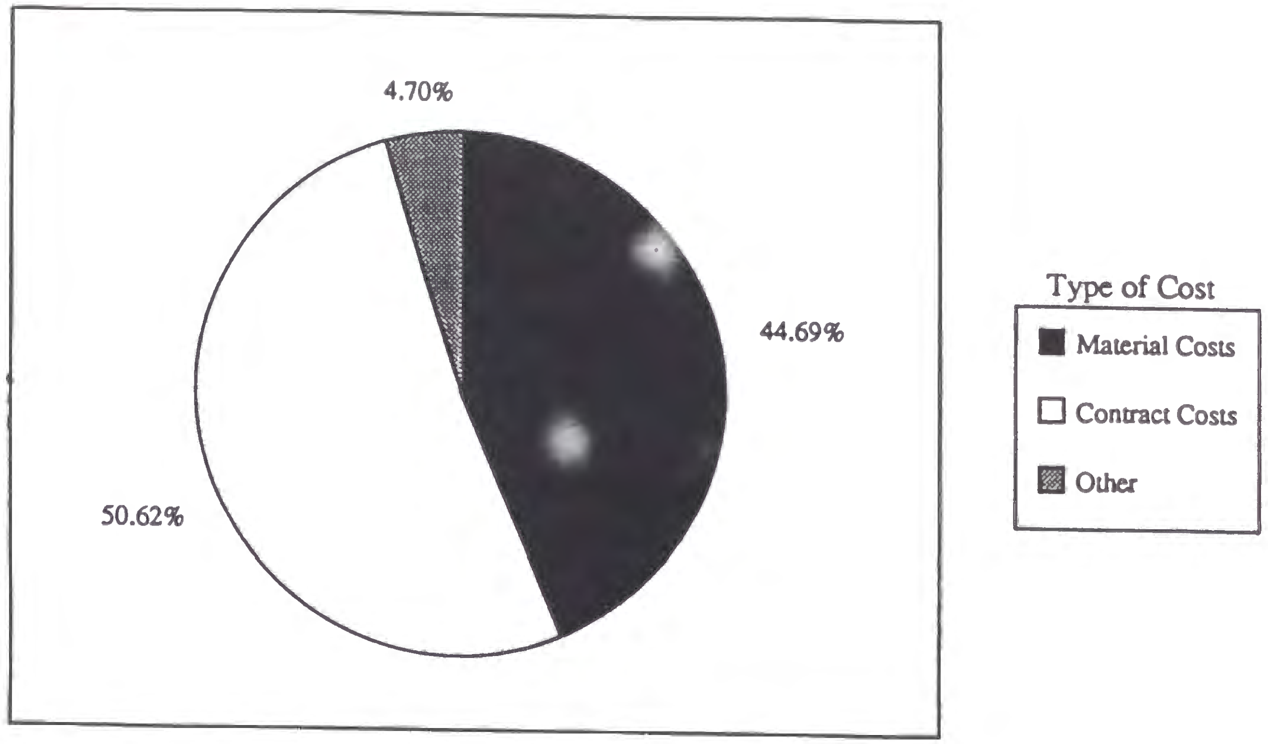


Figure 5.39: Capital cost distribution for pipeline construction

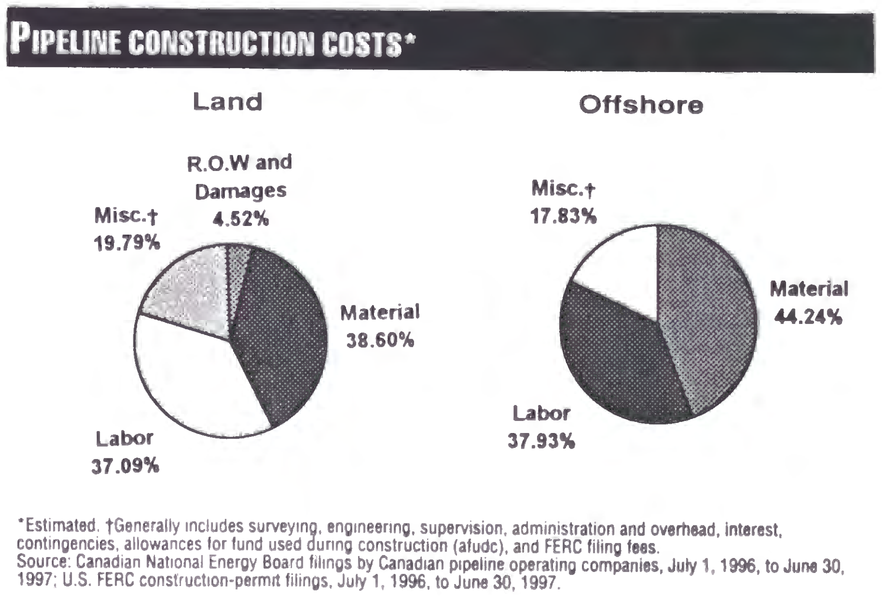


Figure 5.40: Pipeline construction costs (Oil and Gas Journal)

Cost percentages of compression vary from mainline costs. The material costs are approximately 2 to 2.5 times the contract costs, but would change somewhat due to the size of units and specialty requirements such as noise attenuation and emission control requirements. Figure 5.41 shows the percentage components of cost for larger installations (20 to 30 MW range). Compression costs range from approximately 1250M$US/MW for small units to approximately 700M$US/MW for larger units in the 20 to 30MW range. The estimated costs for compression on the ESFE project is about 990M$US/MW, which is again a conservative rate compared with the published ranges, based on the 22.8 MW compression units considered.

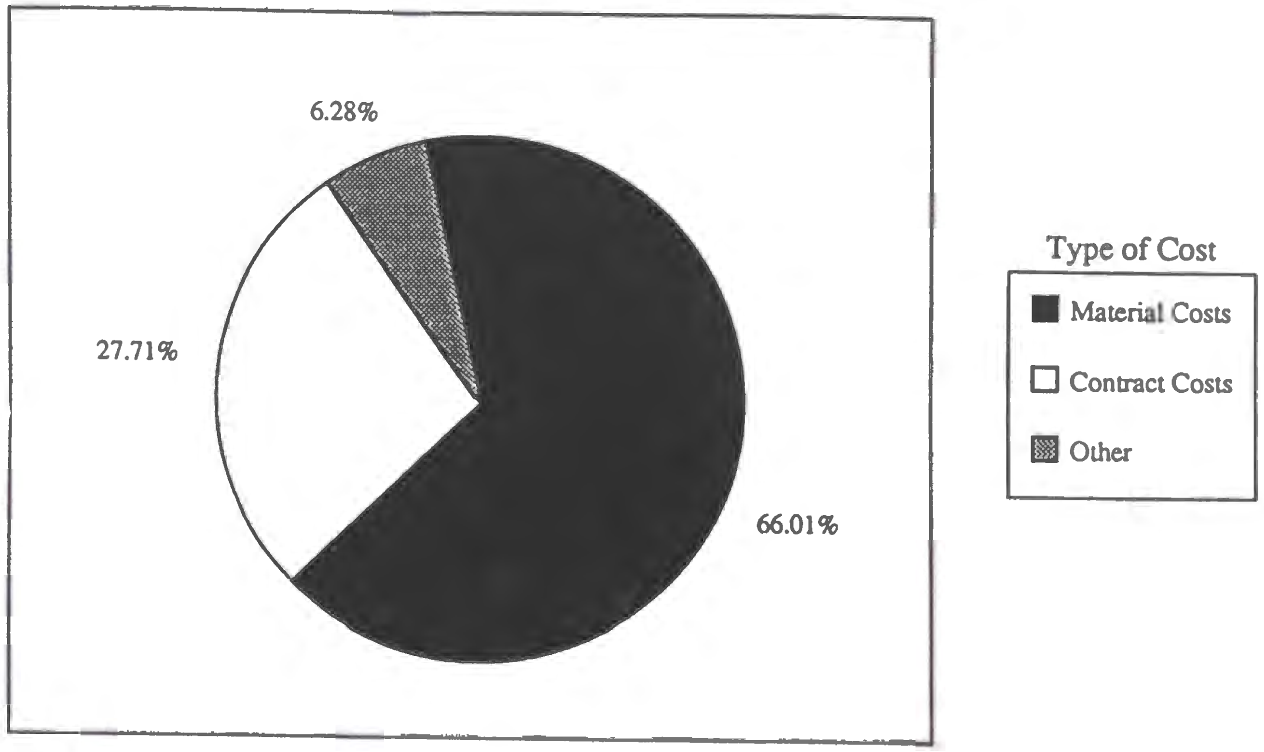


Figure 5.41: Capital cost distribution for compressor station development

6.0 ENVIRONMENTAL CONSIDERATIONS

6.1 GENERAL

There are growing concerns on the part of the public about the construction and operation of pipelines, especially in countries which have a poor record on reliable pipeline operation and environmental protection. These concerns are more acute in areas of greater environmental sensitivity, such as wilderness areas, permafrost regions and in the vicinity of large water bodies. There is also considerable concern near large population centers.

The author has held meetings with, environmental specialists involved in numerous major North American arctic projects and specialists in the reclamation of disturbed permafrost terrain. He has also reviewed the environmental issues specifically related to arctic pipelines. Through this, the author has gained a good appreciation of the main concerns.

Environmental considerations for pipeline projects focus on the following major issues:

- terrain characteristics (topography and geology) - mainly slope stability and changes in slope;

- hydrology - surface drainage conditions, seasonal distribution of precipitation, condition of riverbank slopes and approaches, flood regime of rivers to be crossed;

- soil and subsoil - type, composition, texture of soils and subsoils, erodibility, permafrost condition;

- wildlife - seasonal distribution of wildlife species in the area, identification of presence of critical habitats and endangered species;

- vegetation - amount of clearing required; extent of alteration to natural vegetation during pipeline construction; presence of endangered species; effect of introduction of weeds; and

- land use - present land use and changes as a result of pipeline construction, ecological reserves.

- socio-economic concerns of local peoples and stakeholders; positive and negative impact on adjacent communities; safety; tourism.

Consideration of these issues in the early stages of route planning may significantly reduce potential environmental impacts and the need for special mitigation measures or compensation to landowners (or nearby stakeholders).

6.2 ENVIRONMENTAL CONSTRAINTS

Environmental constraints are identified from analysis of the baseline information compiled for a specific route. They are defined as aspects of the local environmental conditions that will require either route realignment to avoid or extra/special mitigation measures during design and construction to reduce potential impacts to an acceptable level. At this very general level of investigation, the following constraints are known.

The northern parts of the route through the Central Siberian Plateau are subject to boggy permafrost conditions. Winter construction may be necessary in these areas to avoid excessive terrain disturbance. Choice of winter construction may also require advance planning to clear the right of way, strip topsoil and prepare the right of way for trenching.

An area of karst topography occurs near KP 260. Site specific investigations would be required to determine the requirement for specific avoidance or protection measures in this area.

A protected forest zone exists around KP 130-160, and restricted use forest zone lies between KP 410 and KP 430. Specific regulations regarding industrial development and land use in these areas should be consulted prior to more detailed planning.

Lake Baikal is an internationally recognized environmental reserve. In 1979, the Baikal State Nature Reserve was created on the southwest shores of the lake, covering an area of about 1 700 km^2. The area supports 960 species of mammals and 400 species of plants that are found nowhere else in the world. The Russian Federation has developed specific environmental conditions for industrial development near the lake. These must be taken into account during detailed project planning. As well, most areas within 10-30 km of the shoreline are classified as zone 1 water protection areas, with strict limitations on the types of activity that can occur within those zones. While this may appear to differ from existing land use practices, the enforcement of these regulations has become more strict in recent years.

The route through southern Mongolia passes through environmentally sensitive desert areas, with active aeolian features and easily erodible soils. Special measures will be required to ensure stability of the right of way and to encourage revegetation. Several protected areas occur between KP 1200 and KP 1500, including a state nature reserve and an 'environmental museum', and many endangered plants species occur in the KP 1460-1480 and KP 1900-1970 areas. These species include Adonis mongolica, Bungenian iris, Sibbaldianthe sericea and Psammochloa villosa and Haloxylon. An important habitat for cranes, bustards, storks, and vultures occurs along the rivers south of Darhan from about KP 1200-KP 1900. All riverine and riparian habitats assume greater significance in dry desert area and special measures may be necessary to time construction activity to avoid disturbance.

As well, recent archaeological and palaeontological (dinosaur) finds in this area of the southern Gobi desert suggest that special care may be required during trenching to avoid disturbing valuable resources.

6.3 ENVIRONMENTAL PROTECTION

The most effective environmental protection measure is to minimize the total amount of disturbance to the natural ecosystem. The following should be considered during the route selection process:

- Minimize disturbance to sensitive terrain. This includes terrain that is tectonically active, geologically unstable (karst topography), or highly sensitive to erosion. In the vicinity of Lake Baikal, ridges, fault zones and ledges indicate the potential for seismic activity within this area. Options for environmental protection in this situation include route realignment to avoid these areas, or special construction measures to ensure the future integrity of the pipeline.

- Karst topography occurs near KP 260 north of Lake Baikal. This terrain is very sensitive to disturbance and will not provide a stable foundation for the pipeline.

- The pipeline crosses sensitive erosion prone terrain in the Dornogovi Aymak of southwestern Mongolia. The desert in this location is characterized by expansive areas of sandy surficial materials that are prone to wind erosion. Erosion control measures during pipe installation will include avoiding disturbance to active dunes, minimizing the removal of surface vegetation, and applying erosion protection measures such as snow fences or straw mulches on the right of way.

- Minimize the number of water crossings. Fisheries resources can be significantly affected by in-stream construction activities. Fewer river and stream crossings along the route will minimize the potential for adverse effects. Additionally rivers and streams are used as water sources by animals and humans, and changes in water quality can have significant effects on downstream water use. Though proper route selection will assist in minimizing the number of crossings, the proposed pipeline will unavoidably require the crossing of several major rivers. One of the most critical river crossings will be the Kharaa-Gol River which provides important habitat for muskrat and the Daurian hedgehog. The overall objective will be to maintain, in the short term, uninterrupted flow in the channel and minimize downstream siltation from in-stream construction activities. This may be accomplished through bank stabilization and the use of in-stream silt fences. Construction should be timed to avoid fisheries spawning activities. Reclamation will include restoring stream flow and fish habitats through restoring the banks, approaches and stream bed.

- Avoid forested land where possible. Protected forests existing between KP 130 and 160 in the Baikal region should be avoided during the route selection process. The constraints to linear developments in type 2 forests located between KP 410 and 430 require further investigation, and may relate to removal of certain tree species considered rare or ecologically important such as the Siberian stone pine. In general, forested areas provide important habitat for native vegetation and wildlife. Further segmentation of habitat through linear developments may have significant adverse effects on many species that are currently considered rare or threatened.

- Minimize the length of pipeline located in wetlands. Wetland areas increase the complexity of construction, thereby increasing construction cost and time. Wetlands play an important role in the natural ecosystem, particularly for fish, birds, small mammals, reptiles and amphibians. Drainage patterns can be adversely affected by pipeline construction, and are difficult to replicate during the reclamation phase.

- Consider that more information may be required to confirm routing of the proposed pipeline route in the area of the Baikal State Natural Reserve, the Bogd-Khan-Uul natural reserve near KP 1500, the environmental museum near KP 1410, and type

1 protected areas along the shores of Lake Baikal. All these natural reserves and protected areas have been designated as such to protect unique flora and fauna and usually restrict industrial developments of any type within their boundaries.

- Avoid disturbance to protected habitats of endangered animals and wildlife species, identified in Section 6.2. Usually this will require mapping the location of critical habitats and avoiding it wherever possible during the route selection process. Habitat fragmentation is the greatest concern with respect to wildlife habitat, therefore the pipeline should be routed wherever possible to avoid bisection. Construction near wildlife habitat should be timed to avoid critical activity periods such as breeding or nesting to reduce the overall effect of sensory disturbance. For migrating species such as the goitred gazelle and the musk deer, construction should be timed to avoid migration periods, and ancillary facilities should be located away from the migration route. Rare plant surveys along the proposed corridor in critical rare plant habitat will ensure that rare plants may be avoided during construction.

- Avoid disturbance to sites of archeological or historical value in the Dornogovi Aymak. This will require initial site surveys to determine the presence of archeological features along the proposed route, and supervision during construction to ensure that historical artifacts are preserved.

6.4 REGULATORY PROCESS

The proposed pipeline route passes through three national jurisdictions and will be subject to three essentially separate regulatory processes leading to the approval of the pipeline route and issuance of required permits.

All three jurisdictions require the proponent to undertake an environmental impact assessment and to interact with the regulatory authorities in a phased manner during the preliminary and design stages of the project. The requirements are based on the principle that the level of detail of the information provided increases as the pipeline route becomes better defined and as the technical aspects of the project are confirmed. The following subsections outline the general regulatory review process for pipeline projects in Russia, Mongolia, and China.

6.4.1 Russian Federation

Over the past five years, the Russian Federation has gone through a period of rapid change in environmental regulation. New laws and regulations have been introduced to replace those of the Soviet era, but there are still overlapping mandates among agencies with environmental responsibilities, lack of interagency cooperation and lack of regulatory clarity. The basic principles of a 'command and control' economy prevail, and the state has strict procedures and requirements that must be met or approvals will be denied. The Russian process has two distinct stages - pre-design (or pre-project) and design (or project). Project implementation is contingent upon receipt of environmental approvals at each stage.

In the pre-design stage, the Declaration of Intent sets outs out a prescribed package of information required for general approval to proceed. It also includes environmental information that describes the project area and the environmental constraints that should be taken into account during project planning.

The Technical and Economic Overview (TEO) of Investment provides more detailed technical, economic and environmental information about the project, specifically in the consideration of a number of project alternatives. Conclusions are drawn about preferred alternatives and environmental information is presented in the form of a preliminary impact assessment to substantiate the preferences expressed.

In the design stage, the TEO of Construction is prepared, presenting detailed technical and economic information for the chosen alternative, including a full environmental impact assessment. Following State Environmental Review of the documentation, the project may proceed to implementation if a 'positive conclusion' has been received.

The pipeline route passes through the Irkutskaya Oblast, which also contains the Ust'-Ordinskii Buriatskii Autonomous Okrug. Then the pipeline passes through the Buriatiya Republic before entering Mongolia.

Responsibility for issues concerning the use of natural resources, environmental management and environmental safety within each of these 'subjects' is shared jointly between the Russian Federation and the krai or oblast (Article 72 of the Constitution of the Russian Federation). The laws of the Russian Federation apply throughout, but subjects of the Federation may also enact their own laws and regulations in areas of joint jurisdiction, provided they do not contradict the Federation legislation. Therefore, both federal and regional requirements must be complied with throughout the project development. For example, the administration Head of Khabarovsky Krai adopted the Resolution No. 116 of 21 February 1994 "On Regulation of Land Relations". This document sets rules for land allocation until the new federal Land Code is adopted.

Clearly, the project will need to understand and interact with both federation and local authorities on matters of environmental protection, routing, land allocation, safety, and permitting.

Table 6.1: **Russia - Regulation and Permitting Process Relating to Potential Oil and Gas Pipeline Corridors**

Project Stages	Responsible Authority	Timing
Pre-design stage • Declaration of Intent • Environmental Substantiation of Investment (TEO of Investment) • Application for Preliminary land allocation	• Regional executive authorities • State environmental review • Regional executive authorities and Roskomzem (State Committee for Land)	Up to 1 year
Design stage EIA as part of TEO of Construction	• State environmental review.	No more than 6 months
Application for environmental permits • special water use • air emissions • waste disposal • water discharge • comprehensive nature use	• Roskomvod (State Committee for Water) • territorial branch of Minprirodi (Ministry of Environmental Protection and Natural Resources)	One to three months

The Russian Federation has a clearly defined set of environmental standards, which are as tough as any commonly encountered in the G7 countries, for example. In the past, enforcement of these standards has been poor, but this is changing, particularly with regard to projects with significant foreign investment.

6.4.2 Mongolia

The People's Republic of Mongolia is a former Soviet satellite, and its regulatory system has been inherited from that time. Mongolia is currently patriating its legislation and going through a period of regulatory review and change. In 1995, nine new environmental laws were promulgated, however, the basic principles and project stages outlined above for Russia still apply. The time for completion of the regulatory review process is often a concern for proponents. In transitional economies such as Mongolia, human resources are often taxed to their limits in meeting review deadlines, and backlogs and delays are common.

Within Mongolia, the pipeline will traverse three Aymaks - Selenge, Tuve and Dornogov, and possibly part of a fourth, Undgov. The line will also pass through the Capital District of Ulaan Baatar. So, there are several different administrative units for negotiating land allocations and permits.

6.4.3 China

Environmental review is a clearly defined requirement for any construction project in China. Authorization to proceed is not given until approval has been granted for the environmental impact assessment of the project, which must be carried out as part of the 'feasibility study'. Depending on the scale of the project, the review may take place at local or national levels. A pipeline project that crosses several provincial boundaries is automatically reviewed at the national level. In addition, all contracts for construction projects involving foreign investment must conform with national and local environmental protection regulations.

As in Russia, local governments may augment national legislation with local or regional regulations, provided there is no conflict between the two. The environmental review process in China has four primary phases, shown in Table 6.2.

Requirements for environmental impact assessments are set by the National Environmental Protection Agency. Detailed regulations have been developed for the environmentally sound management of major construction projects.

TABLE 6.2 Environmental Regulatory Process in China

Regulatory Stage	Description
Project Proposal Stage	• brief summary of potential impacts • assessment of current environmental situation • opinions and requirements of the local environmental protection department • description of existing problems
Feasibility Study Stage	• completion of environmental impact assessment • describe environmental condition of the project site • list main pollution sources and pollutants • describe potential impact of project • describe environmental protection measures in design • mitigation plan and budget for environmental protection • existing problems and recommendations
Initial Design Stage	• basis for environmental protection design • sources, types, quantities, concentrations of main pollutants • environmental protection standards • facilities for pollution control • mitigation measures • plans for environmental management and monitoring • budget for environmental protection • existing problems and recommendations
Construction Design Stage	• design for all environmental protection measures shall meet all local and state requirements

7.0 CONSTRUCTION PLANNING AND PROCEDURES

Considerable experience has been gained in the construction of major facilities in the remote arctic regions of North America and Russia. The author has learned much of the challenges and constraints to such construction programs through meetings with construction managers and reviewing reports and papers. This section presents some of the ways in which pipeline construction in permafrost regions is planned and some of the unique procedures employed.

7.1 CONCERNS

7.1.1 Preservation of the Permafrost Regime (if Practical)

It is imperative to preserve the permafrost regime for any construction and operations of pipeline or other facilities, if practical. To ignore this requirement is almost an assurance of unacceptable differential settlement and associated water ponding or flow that is likely to result in severe difficulties and damage to the installed facilities.

Any disturbance to the ground surface by equipment or vehicular travel, clearing, grading or placement of buildings directly on the surface will alter the geothermal regime and initiate the thaw settlement. Good construction practice in permafrost regions is to attempt to control the geothermal impact to the minimum of acceptable levels.

The adopted construction practice will involve clearing, grading, the use of snow/ice roads, backfilling and special design of buildings and roadways. Buildings on permafrost must be elevated by piles or by gravel pads to reduce the thermal degradation. A variety of pad concepts can be utilized depending on the type of facility and the ground conditions. These include:

- considerable thickness of gravel,
- gravel combined with insulation, or
- gravel with some piping, to intercept heat from the building and to allow freeze-back of any summer thawing of the ground.

Similar techniques are applied to permanent roadways or utility infrastructure but must address additional work such as culvert freeze-off that can result in extensive ponding of surface water. Techniques covering pipeline construction are addressed in the following sections.

7.1.2 Accessibility and Logistics

The overall construction plan must address the requirement to extend or upgrade the existing transportation infrastructure. The type of upgrading will be determined on whether the requirement is based on a single season construction activity or a multiple season or yearly requirement. For single season requirement snow roads will probably suffice. Whenever the requirement extends over a number of years and represents considerable usage, it would probably merit consideration of more permanent facilities.

Compressor stations, operations and maintenance facilities and operations headquarters should be constructed close to existing roads and infrastructures to minimize the length of access roads and travel time.

As well as ground access, it will be necessary to provide landing strips for small aircraft and helicopters during the construction period which would be available for operations and maintenance activities.

Construction of the Ust'-Kut to Tianjin Gas Pipeline requires the transportation of over 2.7 million tonnes of materials, equipment and supplies which can be broken down as follows:

(i) Mainline	Tonnes
Pipe	2,100,000
Valves, fittings	40,000
Construction Equipment	80,000
Fuel	150,000
Camp Facilities	10,000
Cement, rebar, skids	140,000
Miscellaneous	150,000
TOTAL	2,700,000

(ii) Stations (19 Compressor Stations)	Tonnes
Mechanical	2,000
Construction equipment and camps	1,500
Pipe and valves	1,000
Civil and electrical equipment	4,000
Fuel and supplies	3,000
TOTAL	11,500

In order to transport these large quantities efficiently and economically, maximum use should be made of existing transportation systems and facilities. Taking into consideration river and land travel restrictions, detailed planning needs to be undertaken to ensure that equipment and materials are moved to the staging areas well before the start of construction.

Stockpiles sites should be located as close to the right of way as possible and would be used for one year only. Stockpile sites would not require permanent equipment or accommodation. In Russia and Mongolia the sites will be at the compressor stations while, n China, several stockpile locations will be needed besides the compressor station locations. These sites would be close to existing roads.

7.1.3 Seasonability of Work Periods

The construction scheduling of any pipeline project must satisfy a range of seasonal constraints and other basic objectives relating to productivity and activities dependencies.

Basic objectives will typically relate to achieving maximum production, while satisfying environmental concerns at the lowest costs. Cross-country transmission pipeline construction within permafrost and muskeg/swampy regions, will typically be undertaken during the winter season in countries exposed to sub-zero climates. After freeze-up, construction within the areas of muskeg conditions can be undertaken utilizing only slightly modified procedures, thus assuring cost effectiveness. Within permafrost areas it is essential to work during freeze-up to assure the integrity of the terrain.

In either case, the winter work season represents a duration of 90 to 150 days. In the lower latitudes it is common to use suitable tracked vehicles to pack the snow early in the winter, thus promoting greater frost penetration into the ground. This has the effect of extending the work season. Short extensions are sometimes achieved by working only during nighttime. Under such circumstances, a pipe laying duration of 90 days is typically assured, usually from late December to mid-March.

In permafrost areas the window is usually considerably longer due to the cooler temperatures and the underlying frozen terrain. Typically the work window ranges from late November to late March/early April.

However in all cases, the actual work days can be affected by climatic conditions such as extreme precipitation or temperature cycles.

7.1.4 Resource Utilization

Resource utilization for a long distance pipeline is very important to the resource requirements, project duration and costs. Although personnel can be mobilized or demobilized quite readily, this may not be the situation for equipment and camp facilities. The construction scheduling attempts to balance the winter/summer work periods to achieve the maximum utilization of equipment and continuity to the personnel utilization. This requires an allowance of sufficient time for the transfer of equipment, camps and other plant from the job site to the all-season transportation infrastructure system before thawing occurs.

If suitable roads, railroads or waterways, exist very good utilization of resources can be achieved. If they do not exist utilization would be reduced substantially and result in considerably greater costs. The assignment of spreads and the direction of work must be planned to assure the minimum movement of resources.

The number of workers required to execute a project of this magnitude is enormous. The number of workers vary between winter and summer seasons. Each season may require a different distribution of labour and manpower peaking due to climatic conditions and the available work season.

These influences create significant peaks and troughs in the requirements and create a situation of necessitating seasonal mobilization and demobilization of many of the field supervisory and labour personnel. This situation would likely be aggravated by having limitations imposed on cross-border utilization of workers.

Attempts to balance labour requirements to the extent possible, help to reduce peak requirements of workers, better scheduling of work activities and reduced potential for work interruptions. The balancing of labour is best achieved by advancing as much work as possible on a seasonal or yearly basis.

Clearing and rock grading is best achieved by advancing the activity a full season or year ahead. If the pipe laying is defined as winter work, all clearing would logically be undertaken only during the winter season. The scheduling of rock grading activities may provide greater flexibility depending on access and the continuity of the rock formation. Other activities that should be seasonally advanced are borrow site development, pad development, camp erection and all-season road development. These and other related activities provide a better insight to work conditions and potential problems. In addition, they eliminate delays that may occur for many reasons, if scheduled during the same season as the mainline construction, thus assuring conditions that allow a smooth startup of the various pipe laying activities.

Creating an execution plan that attempts to reduce or eliminate peaks in the resource requirements assists directly in controlling the buildup of labour on the job site. Right of way preparation should be well advanced before any pipe is introduced to the right of way. Subsequent operations should be phased to assure the most effective utilization of resources. Depending on the work quantities, some crews can handle more than one activity, i.e., the lower-in crew could also be involved in tie-in operations.

7.1.5 Availability of Borrow Materials

The necessary borrow materials will range from materials suitable to provide select backfill, to gravel for the construction of pads or the production of swamp weights. The preferred borrow locations will be depend on terrain conditions and the locations of the facilities.

It is common practice to undertake sufficient field investigation to identify and quantify suitable sites before any construction is undertaken. Techniques that are commonly utilized consist of air photo interpretation of local land formations followed by field confirmation of this interpretation. Such work is not only justified for construction planning but is cost effective for remote or undeveloped areas.

Similar techniques are usually applied to identify subsurface rock and are combined into a common program.

7.1.6 Availability of Suitable Local Construction Equipment and Skilled Labour

A pipeline system crossing through a number of countries is most certainly to be confronted by pressures from the countries involved to maximize their participation in the supply of services and materials. Although compromises will be necessary they should not be allowed to interfere with the optimum lay rate for the pipeline. Any significant reduction in

production will require the utilization of more resources or result in an extension of the project duration. Either of these situations would increase the overall investment costs.

A project of the magnitude of the ESFE pipeline system will require very large quantities of workers and services supplied on a regional basis. The demand is likely to strain or exceed the availability of such resources and could require mobilization on a national basis.

The activities that typically control the lay rate are welding or ditching, with the latter applying more to permafrost soils or rock. Jumbo trenchers developed for the arctic are the only trenchers that have demonstrated the capability of trenching in permafrost without blasting assistance. These machines have the depth and width capability to handle 56 NPS pipe. Using suitable teeth, they are capable of trenching 1.5 km per shift.

Using trenchers provides the additional advantage of creating spoil that is suitable for backfilling without the requirement of pipe coating protection or select backfill.

These trenchers have provided sufficient advantages that they are commonly used during conventional summer construction if the terrain conditions are suitable.

For a pipeline system of this diameter and length, the cost of pipe represents an enormous investment and the best economics are obtained by utilizing the highest grade pipe that is practical. Currently this is probably X80, although testing of higher grade pipe is being undertaken. Automatic welding should be the only type of welding considered for this application. It provides the best controlled conditions and weld quality for either summer or winter construction. Most of the welding crew do not have to be qualified welders and there is a reduction of the number of welders required per spread.

Other equipment has less effect on the work and are therefore not addressed.

7.2 PLANNING

7.2.1 Seasonal Constraints

The seasonal constraints are basically to avoid long term environmental disturbance by timing the construction period to take account of terrain conditions. The project can be installed in a more efficient manner. The most significant constraints relate to the sensitivity of the permafrost terrain and the numerous swamps along the northern portion of the pipeline route. It is necessary to construct these segments during winter when the equipment can be supported on frozen ground. The majority of all northern construction in North America and Russia is constructed in winter.

Some specific environmental constraints along the proposed pipeline route were presented in Section 6.

7.2.2 Pre-Pipe Laying Activities

Certain activities that should be undertaken prior to pipe laying operations increase the efficiency of construction and results in overall savings in costs. Activities/scheduling that have proven beneficial include the following:

- Identify and test borrow sources well ahead of mainline construction. These sites should be developed early and used as appropriate to satisfy the enormous borrow requirements of the project.
- Pre-clear pipeline rights of way. This allows a better assessment of terrain condition and of work quantities such as grading work, excavation, buoyancy control, drainage and erosion control requirements, etc.
- Undertake rock grading well ahead of construction.
- Develop compressor station sites early so that they can be used as stockpile, camp, double-jointing and fabrication sites.
- Accelerate frost penetration into working ground surfaces to allow an earlier construction start on the frozen ground in the northern sections. This is achieved by "walking" wide pad tractors or flex-tracked vehicles over the ground surface after early frosts.
- The ditch line is usually protected from seasonal frost penetration by removing excess snow from the spoil side of the right of way and roached over the centerline until the trench is to be excavated.
- Develop access roads, bridges and other infrastructure early to ensure pipeline construction will proceed on schedule.

7.3 SCHEDULE OF WORK ACTIVITIES

The proposed construction schedule for the entire pipeline project is to take three years. Figure 7.1 presents the schedule for the pipeline construction. Figure 7.2 presents the schedule for the main river crossings, which would likely be constructed as separate contracts. All other smaller river crossings as well as road and rail crossings will be performed by the main construction spreads. The schedule for the construction of the facilities is presented on Figure 7.3. All major components of the mainline pipeline construction activities are discussed in detail in the following sections, especially as it relates to the winter construction and permafrost conditions.

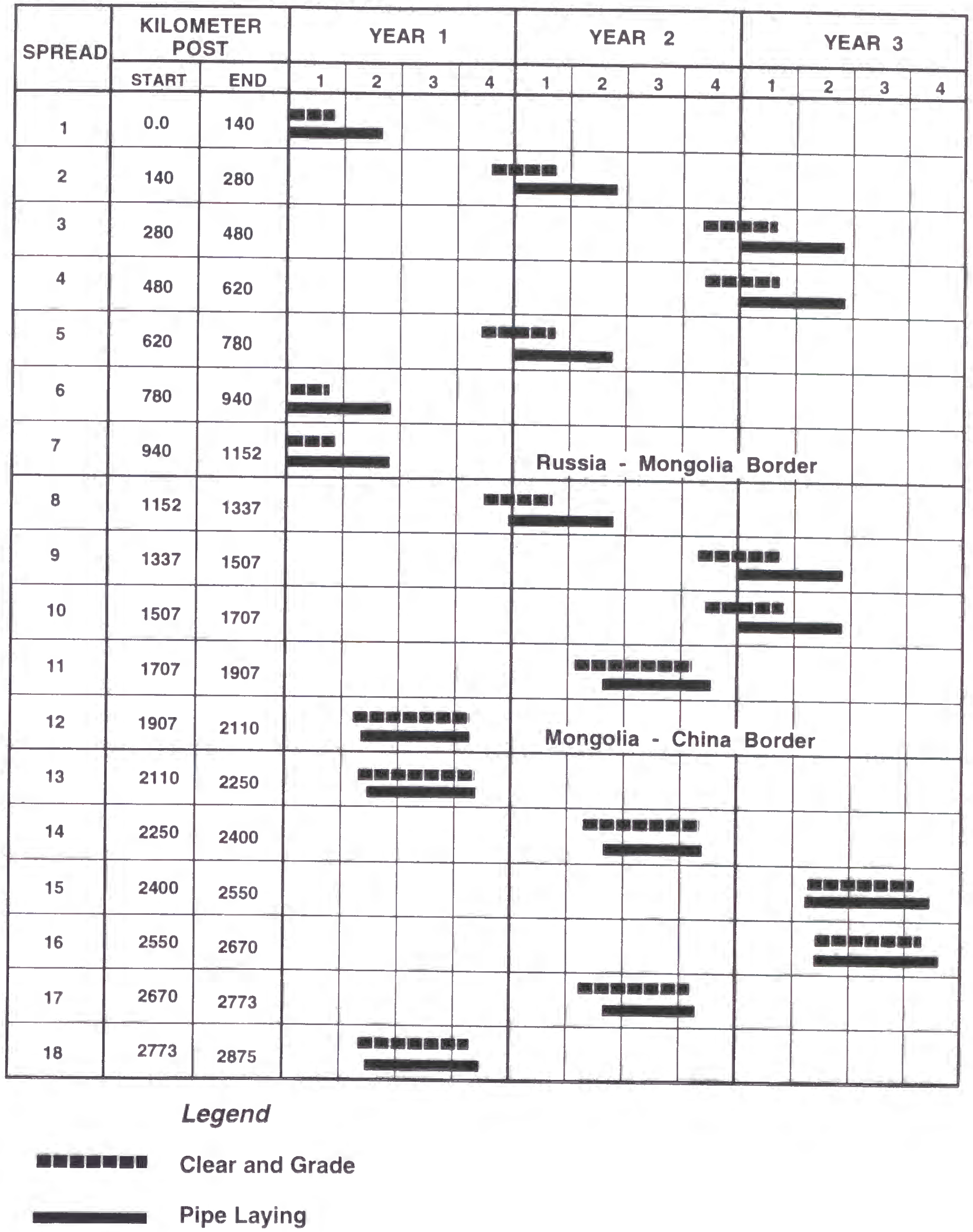
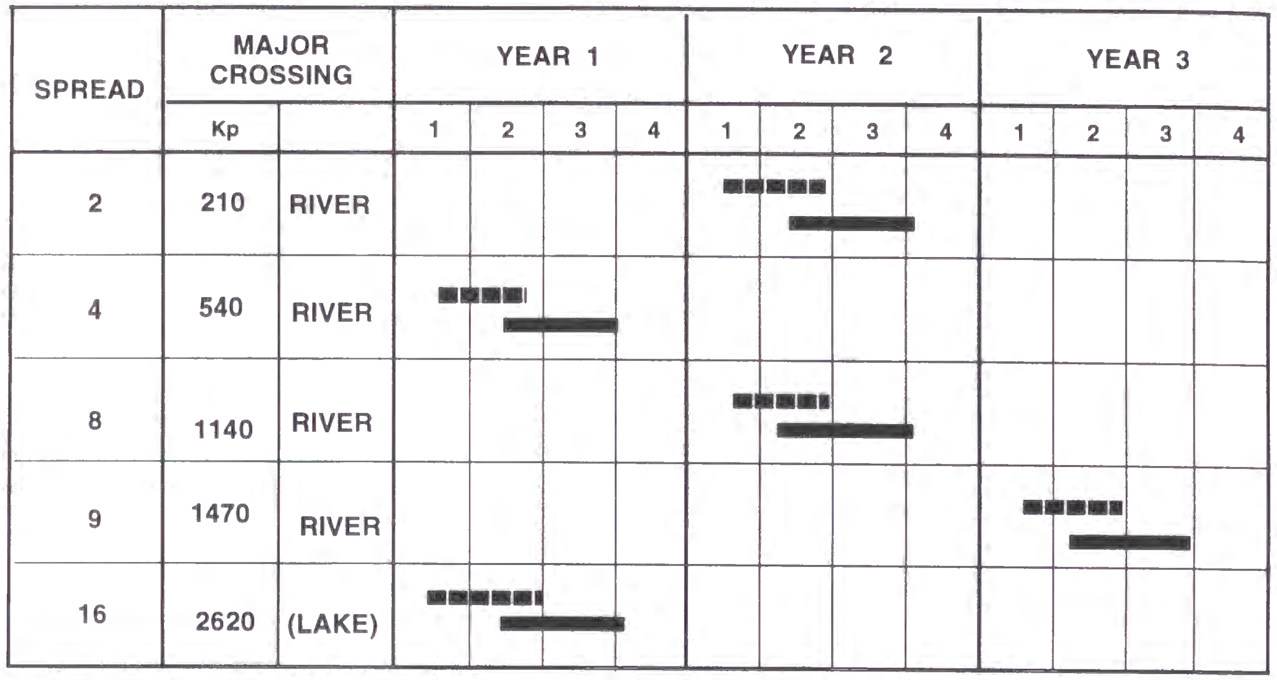


Figure 7.1: Pipeline Construction Schedule

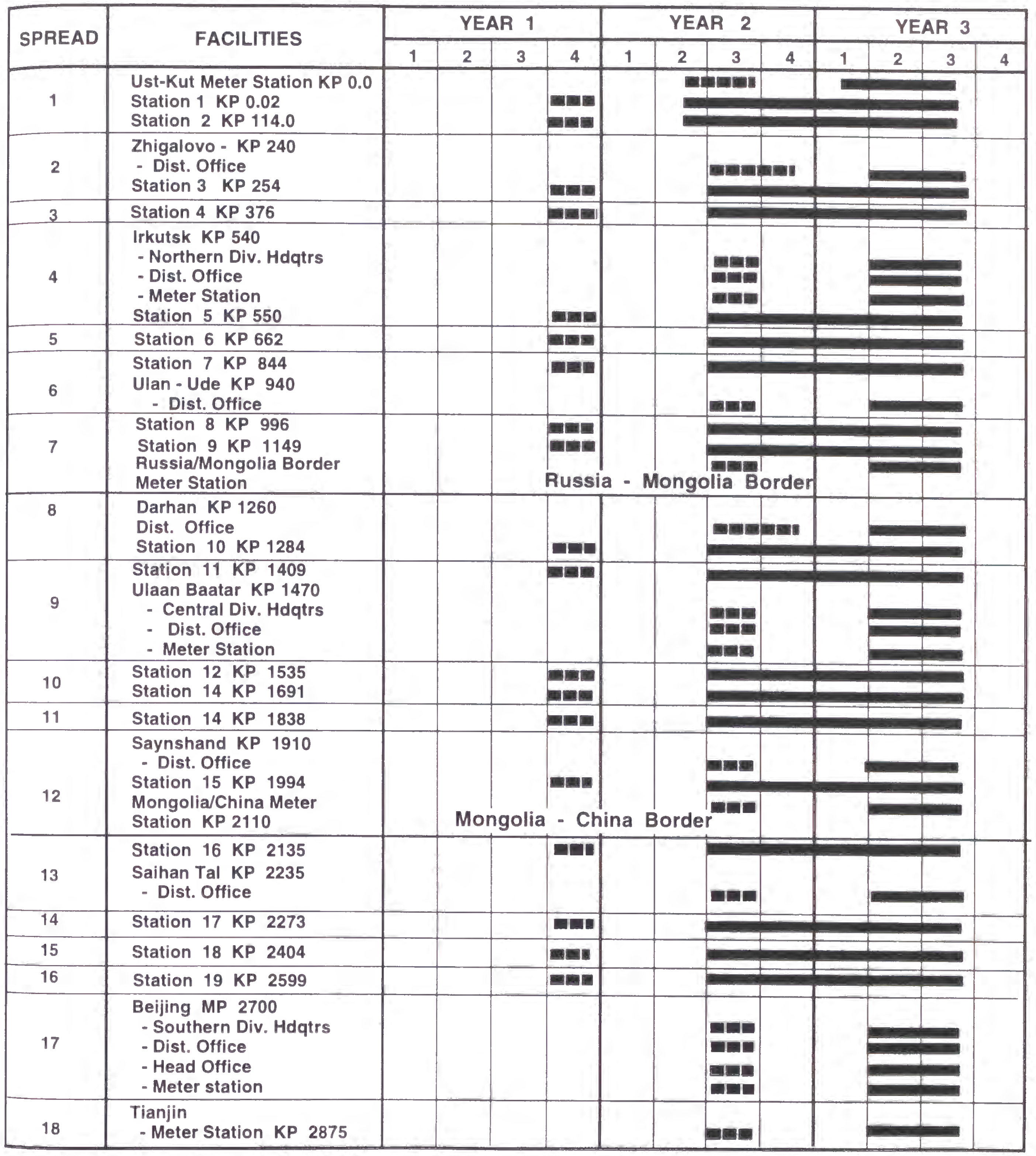


Legend

Clear And Site Preparation

Pipe Installation

Figure 7.2: Major Crossing Schedule



Legend

Site Preparation

Construction

Note: The communications system will be constructed during the same period as the other facilities.

Figure 7.3: Facilities Construction Schedule

7.3.1 Right of Way Preparation

7.3.1.1 Clearing of the Right of Way

The construction right of way will be in the order to 30 to 35 m wide to allow sufficient space for all construction activities. Figure 7.4 illustrates the width requirements, which will generally be greater for winter spreads due to snow banks.

To provide a work surface suitable for the pipe laying and allied operations, it will be necessary to clear the work areas of all trees and brush. Grubbing will generally be conducted on that portion of the right of way on which ditching equipment will travel or as part of normal grading. Construction areas requiring clearing will include:

- pipeline right of way
- access roads
- compressor station sites
- meter station sites
- borrow pit areas
- stockpile areas
- airstrips and helipads
- administration centre sites
- communications sites

The clearing procedure will depend on whether the timber must be salvaged or otherwise disposed of.

- Where specified, merchantable timber will be cut, delimbed and stacked for subsequent removal or used for riprap (corduroy) where it is necessary to form a stable work and travel surface.

- If timber is not to be salvaged it will be machine cleared using suitable timber dozer blades.

- Within permafrost regions, the dozer blades should have surface rails to minimize surface disturbance.

- All other timber, brush and stumps will be burned or disposed of in accordance with applicable regulations. Burning sleds can be utilized if considered necessary to reduce the potential for permafrost degradation.

7.3.1.2 Snow/Ice Roads and Ice Bridges

Cut grading is generally unacceptable in most permafrost regions and fill grading is utilized to achieve a satisfactory work surface. Fill grading can be undertaken by suitable soils or by snow and ice.

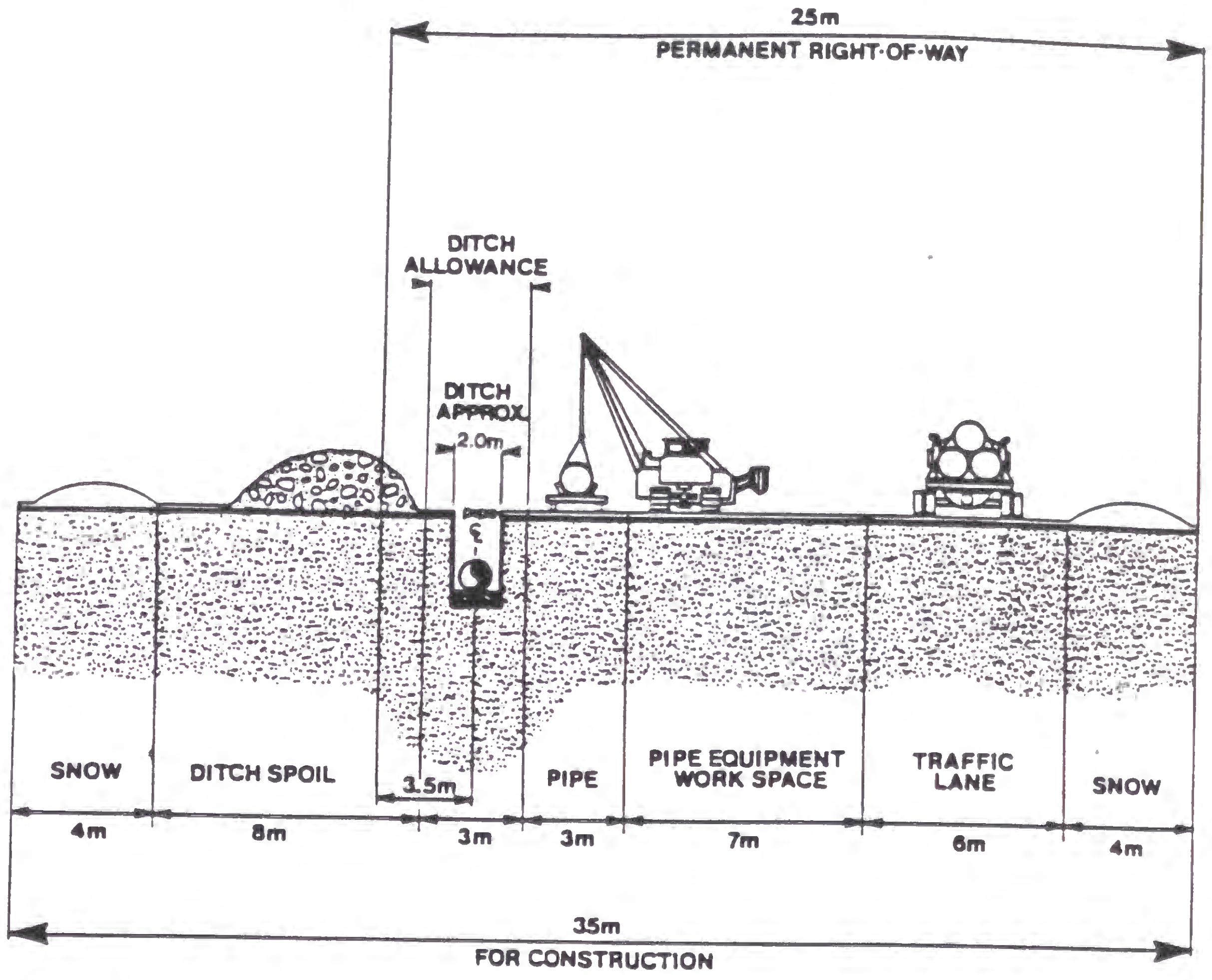


Figure 7.4: Construction Right-Of-Way configuration

Special grading and temporary crossings will be required at roads, rivers, streams and ravines to permit passage of construction equipment. Within permafrost areas, natural or artificial snow can be utilized to prepare grade for streams or road crossings and other surface irregularities.

During grading operations care will be required to ensure that excavated material or other debris does not interfere with streams and drainage.

Within the permafrost regions, it is necessary to protect all working ground surfaces with some type of protection against significant disturbance. The method adopted will likely depend on the location, potential disturbance and the sensitivity of the permafrost to such disturbance. It is common practice to pack all naturally distributed snow within the right of way. This is usually satisfactory to protect against occasional traffic. To protect against high traffic disturbance in moderate - high ice areas, snow or ice roads should be developed. Various techniques are available but all generally include the collection and packing of now, scintering or beating of the snow (similar to rototilling) and repacking. This increases the density and hardness of the snow and results in much improved road requiring less maintenance. This can be repeated a number of times and the surface can be protected by spraying with water.

Wherever ice roads are required for crossing of river crossings, construction should be commenced as soon as possible to assure an ice depth satisfactory to support the heavier foreseeable loads. This could require 24 hour flooding to assure that construction is not delayed by access.

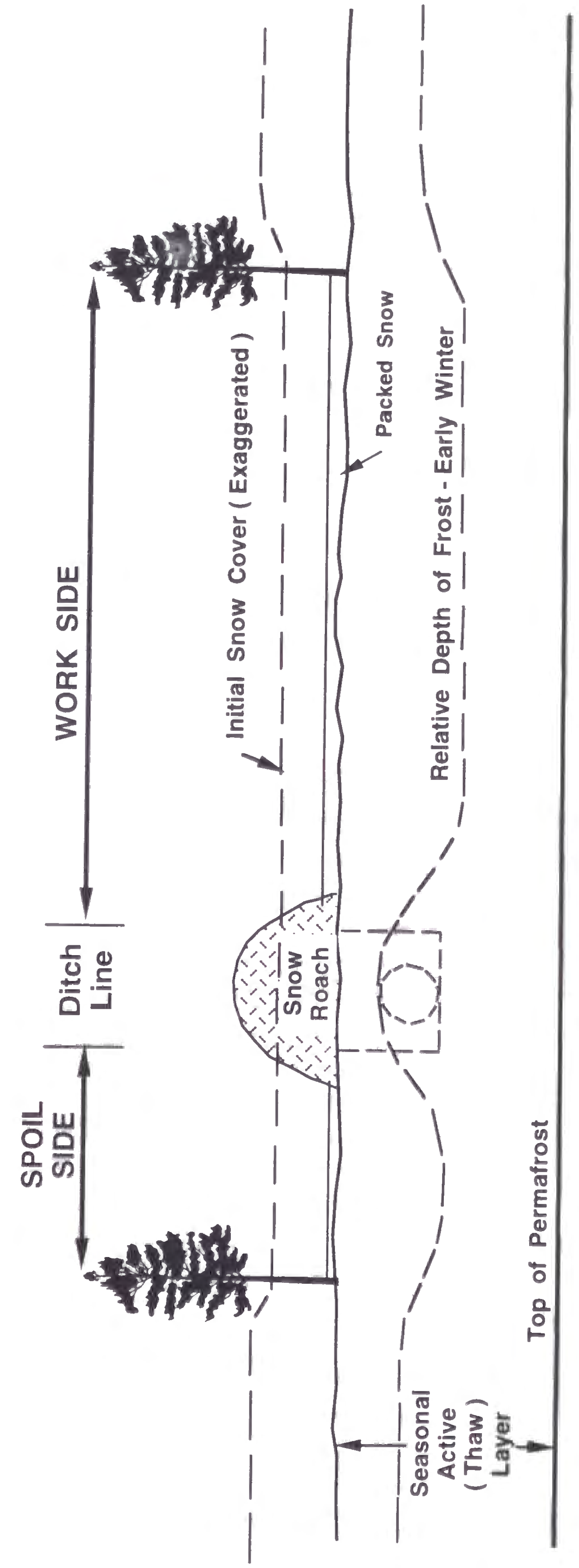
7.3.1.3 Ditch line Roaching

This is a technique utilized to delay frost penetration along the ditch line. During winter construction it is common practice to blade some of the initial snow cover to a position over the ditch line to reduce frost penetration. This reduces the difficulty of trenching as the degree of difficulty increases with colder frozen temperatures. Elsewhere on the right of way, snow is cleared or packed to promote frost penetration as soon as possible. Within permafrost regions snow is moved from the spoil side of the ditch line and the working side is packed for use as snow roads or a base for snow road construction.

Figure 7.5 demonstrates the relative depth of frost penetration and the effect of the roach.

7.3.2 Pipe Stockpiling, Hauling and Stringing

Stockpiling of pipe should occur well in advance of the start of construction. The stockpile sites will depend on the primary mode of transportation (rail or barge) and the availability of sidings and storage space contiguous to rail or barge off loading facilities. Use of such sites can eliminate the need for additional handling of the pipe, that could be required for other storage sites. Any extra handling represents greater costs and may require the preparation of other sites.



Notes: 1) Trees cleared one year prior to pipelaying season.

2) As early as possible in winter (November) snow should be packed on the WORK SIDE, using wide - pad dozers over soft swampy ground. This will promote greater frost penetration for better support of traffic and heavy equipment. Snow surface can be strengthened by spraying water on surface so it can freeze.

3) At same time, blade snow from SPOIL SIDE into roach over the ditchline This will reduce the frost penetration to facilitate trenching.

Figure 7.5: Preparation of Right - of - Way for winter construction in permafrost regions

The pipe weight restricts load limits to two joints for conventional flat bed or pole trailers to stay within normal limits. Off-loading is undertaken by the use of cranes transferring the pipe onto suitable sleepers (timbers) to keep the pipe off the ground. For pipe of this size, padding such as sandbags should be used to prevent damage to the pipe and coating due to excessive loading. Pipe should be stacked in a pyramid configuration to assure load distribution.

Pipe will be transported by truck from the stockpile sites to the right of way. The travel surface and grades should be constructed to accommodate an efficient trucking operation. Construction of these surfaces will reflect the time of year, terrain conditions and access to the right of way.

Appropriate lifting tackle, padded bunks and covers should be used to avoid damage to the pipe and coating.

7.3.3 Pipe Bending

The pipe will be bent to fit the contour of the ditch bottom using the cold, smooth bend method, except in special situations where shop-fabricated bends are required. All bends will conform to the requirements specified by the applicable standards. Bending of pipe will be performed using bending machines and internal mandrels.

Bending could prove to be exceptionally difficult for high grade, large diameter pipe and commercial equipment capable of handling such loads are unlikely to exist. Development of the required bending machines would not, however, be difficult. There should be the capability of simply increasing the power of currently available machines.

7.3.4 Welding

The selection of the welding techniques will likely be disputed to protect the interest of participant nations. However, automatic welding is the logical method of pipe jointing. The economics involved in a transmission system of this magnitude dictates the use of high yield pipe to assure acceptable levels of cost of service and market pricing. Various automatic welding types and manufacturers are available and serious assessment should be undertaken to select the best combination of quality and cost. Considerations include the acceptability of a copper backup ring and only external stringer hot pass metal deposition versus the combined internal and external passes to achieve the root disposition. Different manufacturers utilize a dispersed number of weld heads for filler bead deposit and this can influence the production achievements.

Whatever the selection, a procedure qualification must be developed and qualified prior to the start of any production welding. This welding procedure will set parameters for matters such as process, diameter group, wall thickness group, joint design, type of filler metal and number of passes, electrical characteristics, number of welders employed, time lapse between passes, preheating requirements, lineup clamp removal, speed of travel, appearance of the weld, ambient temperatures and weather conditions.

Although the use of tents or windbreaks is standard practice for automatic welding, it will be essential for winter working conditions. The enclosures developed for winter conditions

provide heating, lighting, air transfer and power outlets. The enclosures protect the workers from the outside chill factors, the critical work area from blowing snow or rain, and to some extent reduces the cooling of the pipe after preheat. All of these factors improve the quality and the productivity of the work. Following completion of welding of girth welds, insulated blankets are positioned to control the post-weld cooling.

7.3.5 Trenching

The diameter of the pipe and the existence of permafrost provides the circumstances that are particularly appropriate to the large arctic-type ditching machine. Machines were developed to handle the extreme service conditions involved in the arctic environs. They can be fitted with suitable teeth matching the soils condition and can obtain remarkable production rates under severe conditions. These ditchers have the capability of handling the necessary ditch widths and depths but also cut spoil which is suitable for backfilling without the need for padding under many circumstances.

Preliminary engineering intelligence indicates the existence of both hard and "soft" rock. The hard rock will undoubtedly require blasting, but it is assumed that the soft rock is rippable and removable by backhoe.

The ditch bottom must be cleared of all debris and must be padded within rock areas, and in frozen soil excavated by hoe or other possible irregularities exist.

7.3.6 Lowering-in

Prior to the lowering-in of pipe, the ditch will be cleared of debris and other foreign material. Where the ditch bottom cannot be cleaned and finished to provide a smooth, even surface, a minimum of 0.2 m of bedding material will be placed and spread evenly on the ditch bottom or pre-installed foam supports will be used.

The lowering-in operation will be conducted by lifting the pipe off the skid supports using sideboom tractors equipped with wide, nonabrasive belts or rubber-tired cradles.

This method will prevent damage to the pie coating. The coating will be electrically inspected and any defects will be repaired prior to lowering the pipe into the ditch.

7.3.7 Buoyancy Control

Buoyancy control will be required for any crossings of water courses or swampy terrain. Weighting techniques typically consist of concrete bolt-on weights, set-on swamp weights or continuous cement coating. Continuous coating is most commonly used for major river crossings that require the pipe to be pulled into place. Bolt-on weights are used most commonly with smaller shallower rivers where the cradling equipment can cross the river and lower the pipe into a prepared trench. Swamp weights are only common where swamp or muskeg contains sufficient peat or vegetation that helps to counteract pipe movement. Determination of the weighting requirements is usually made by field investigation.

The bolt-on weights are not considered a wise choice for pipe of this diameter and weight. The pipe with bolt-ons installed represents an exceptionally heavy load and could present difficulties for side booms working on the river bed.

7.3.8 Tie-Ins

Tie-ins by girth welding will be required to join the welded pipe sections into a continuous pipeline. The sections will vary in length due to the location of roads, railroads, foreign pipeline crossings, river crossings, valve installations and the specified maximum length of sections.

Tie-ins will be made using conventional practices for cutting, aligning and welding the pipe ends. Tie-in welds will be inspected both visually and by nondestructive methods.

7.3.9 Backfilling

The backfill operation will be accomplished by means of bulldozers and auger dozers, morman boards, clams or draglines or both. Since rock, gravel or frozen material found in the spoil may cause damage to the coating, the pipe will be protected by placing a minimum 0.15 m layer of padding material round the pipe, where required. The padding material will consist of processed borrow. Unacceptable spoil will be completely replaced with select granular material.

Where cost-effective, a net-like, polypropylene plastic curtain (Netlon or equivalent), or rock shield will be placed around the pipe for coating protection.

During the backfill operation, the pipe will be visually inspected for damage to the coating. Repairs will be made as required, in conjunction with the backfill operation. Test leads, spaced as required to permit the monitoring of the cathodic protection of the completed line, will be installed. After lowering-in and placement of required padding, the remaining spoil material will be placed over the ditch centerline. Where applicable, soils stripped from the ditch line in the grading operation will be used to form a crown to compensate for subsequent thaw settlement.

Openings will be left in the crown at appropriate intervals and will be stabilized with coarse granular materials to permit lateral surface drainage.

Only those spoils excavated from the pipeline ditch or imported borrow will be used for backfilling. Other spoil from the right of way grading will not be used unless otherwise specified. Excess rock and other unsuitable backfill materials will be removed from the right of way and will be disposed of at approved land reclamation areas.

In areas which are subject to erosion or slope instability, terraces or cross ditches will be constructed to divert surface runoff away from the ditch line to existing drainage systems. All drainage ditches will be maintained and left unobstructed to avoid backing up the water.

On steep slopes, ditch plugs or breakers, constructed with sand bags or possibly foam, will be installed around the pipe in conjunction with surface diversion berms to prevent runoff from following the ditch line.

7.3.10 Special Crossings

The main crossings will be for rivers, railways and, major and minor roads. For the most part conventional design and construction approaches will be adopted. Where appropriate, directional drilled river crossings should be considered; however, such crossings tend to be much more expensive than conventional crossings. The main justification for directional crossings is usually unstable river banks or approach slopes or, particularly sensitive environmental conditions. However, directional drilling may not always be feasible, in terms of the soil and geological conditions.

Railway and major road crossings are usually constructed by boring. Minor crossings can be constructed by boring or cut and cover.

7.3.11 Valve Installation

The valves required on this project are so large and heavy that prefabricated valve assemblies cannot be considered, although some components such as rivers and crossovers could be prepared and moved to site. In North America, it has become common practice to do all such fabrication at site, using contract specialists. This allows the main work crews to position and tie-in each completed assembly. Main line valves would be coated and buried.

7.3.12 Pressure Testing

Following installation of the pipeline segments, prooftesting of the pipe is required to certify the integrity and stress capacity of the pipeline. The plan would define the test segments test media sourcing and disposal, pressure levels and durations.

In general, the program would utilize an approved test medium that satisfies the geotechnical conditions, ground and ambient temperatures at specific sites. Water is the standard test medium but may have to be modified, using freeze depressants or heating, for winter applications.

The project would prepare a profile of the mainline and, using the following criteria, will divide the line into a number of test sections. Test sections will vary in length according to the elevation change within the individual section and location of water sources. The limitations for each test section are as follows:

- At the point of maximum elevation of the test section the minimum test pressure will be that which will produce a hoop stress of 100 percent SMYS.

- At the point of minimum elevation of the test section the maximum test pressure will be the lesser of:

  - that pressure necessary to produce a hoop stress of 110 percent SMYS;
  - that pressure which will produce a deviation 0.1 percent from the straight line proportionally of a pressure versus volume plot.

During winter construction hot water is often utilized as the testing medium. The water is circulated long enough to create temperature conditions that assure that no icing occurs.

The duration of the test will comply with regulations or good practice. Engineers will record and calculate the hydrostatic head calculations occurring during the actual test period. Following successful testing, the test medium will be transferred to the succeeding test section and reused. Any disposal of test water must satisfy any existing regulations but common practice is to filter the water and return it to local water sources.

Testing of the pipeline with additives to depress the freezing point of water will require special procedures to eliminate the depressant prior to releasing the water. On the other hand should air testing or testing with natural gas be undertaken the disposal problems become less onerous. The air can be simply injected into the atmosphere and the natural gas (for natural gas pipeline) can form the linepack for the system. According to some standards and codes, pipelines cannot be tested to higher than 90% of the specified yield strength of the pipe where air or natural gas or where gaseous testing medium is used.

Following testing completion of contiguous segments, the sections will be tied-in. The system will be cleaned of any water using air drying or methanol washing.

Compressor and meter station piping will be tested hydrostatically to a minimum test pressure of 1.5 times the intended maximum operating pressure. The test will be conducted according to the requirements of the latest edition of ASME B31.8 Gas Transmission and Distribution Piping Systems. The test area will be posted and only persons involved in the test will be permitted in the area. The maximum test pressure will not exceed 110 percent of the SMYS. Test pressure duration will be in accordance with applicable regulations.

7.3.13 Pipe Protection Prior to Commissioning

Practices vary and seldom is there any concern to protect the pipe prior to commissioning unless multi-year contracts are involved. Standard practice is to fill the pipeline with nitrogen or some other inert gas.

7.3.14 Right of Way Restoration

Following construction, all surplus construction material, waste and debris will be disposed of as soon as possible. Cleanup of all land areas utilized and the disposal of materials will be carried out in a manner that is required by regulations.

The ground surface will be restored as close as possible to its original state, taking into consideration future drainage patterns necessary to preserve the integrity of the installation. The installation of diversion berms or rip rap to control undesirable erosion.

The final step in the restoration, after all traffic is finished on the right of way, is to spread grass seed and fertilizer. Much research has been conducted in the North American arctic to determine grass species that will develop a rapid cover even in the very harsh climate conditions. Winter seeding has the advantage that the seed can be placed by driving large seed broadcasters along the right of way. Then the seed is in place as soon as the warm weather commences. The snow melt provides good moisture for germination. These

procedures have proven very successful for oil and gas wellsite reclamation and for pipeline rights of way in North America. They have also been demonstrated by North American scientists to be successful in the Russian arctic, on the Yamal peninsula.

7.3.15 Application of Slopes Mitigation Measures

Following cleanup and basic restoration of the surface on slopes, any special slopes mitigation is applied. This will include placement of diversion berms and rip rap erosion protection at the toe of slopes potentially affected by adjacent water flow. Some slopes may require special measures for river engineering purposes if there is a potential for river bank migration.

On certain ice rich slopes, stability concerns may require the placement of wood chip insulation. The wood chips should be obtained by chipping nearby trees, as best as possible to reduce haulage costs. Only, softer evergreen tree species should be utilized for the wood chips. Harder deciduous trees or trees with any rot should be avoided.

The wood chips should be placed to the design thickness and be finished to a good smooth surface. Special protection, such as timber cribbing, should be placed at the toe of slopes exposed to potential water erosion, so that the wood chips are not washed away. Any thermal monitoring installations should be placed immediately upon completion so that the expected initial heat generation due to biodegradation can be monitored.

8.0 COMMISSIONING, OPERATION AND MAINTENANCE PLAN

There are some unique approaches required for the operations and maintenance of pipelines in permafrost regions. The most important relate to the impact of the pipeline on the permafrost and the potential for thaw settlement and slope instability. The author's research and consultations have identified the most important items unique to northern pipelines.

8.1 LINE PACK

On completion of construction of the pipeline, the pipeline must be filled with the gas that is to be transmitted through the system. Any remaining testing medium must be removed ahead of filling the pipeline. Pipeline "pigs" are utilized to separate the gas from the test medium. As well, methane or other suitable additive is injected between two pipeline "pigs" to absorb any water remaining behind the dewatering scraper.

Natural gas pipeline temperatures and line pack volume will vary considerably along the length of the pipeline and with the time of year, particularly in northern areas. In North America, the natural gas is generally one stream of constant specification which will be suitable for transportation at all pressures and temperatures anticipated during normal operations. The natural gas is generally processed to the required pipeline specifications at gas plants prior to acceptance into the system.

8.2 PIPELINE OPERATIONS

Head Offices will be located in each of three countries i.e., Irkutsk, Russia; Ulaan Baatar, Mongolia and Beijing, China. For the purposes of this study it has been assumed that the Beijing office will provide the overall coordination functions for the Company. Each Head Office organization will have a total of 200 people who will have responsibility for establishing policies and procedures for the safe and efficient operation of the pipeline system in that country. This group will include departments holding responsibility for the provision of support services to the field organization. Gas control will be located near the terminus of the system near Beijing.

The field organization will be divided into three divisions. The Northern Division office, located at Irkutsk, will be responsible for all facilities in Russia (north of KP 1152). The Central Division office, located at Ulaan Baatar, Mongolia, will be responsible for all facilities in Mongolia (between KP 1152 and KP 2110). The Southern Division will be located at Beijing, China and will be responsible for all facilities in China (between KP 2110 and KP 2875).

The Division offices will be primarily business offices providing administration and technical support services for all facilities within their jurisdiction. North and Central Divisions will be subdivided into three operating Districts while the South Division will have two districts.

The Northern Division (Russia) will be headquartered at Irkutsk and will have district offices at Zhigalovo ,Irkutsk and Ulan-Ude. The Central Division (Mongolia) will have its District offices located at Darhan, Ulaan Baatar and Saynshand. The Southern Division (China) will have its district offices located at Saihan Tal and Beijing.

Each District will be supervised by a Manager responsible for the day-to-day operations and maintenance of the facilities within his area of jurisdiction. Each District will be manned and equipped to carry out routine maintenance and operations requirements. Contingency situations may require the mustering of additional personnel from other Districts or by local hiring.

Control and monitoring of all facilities will be carried out by the Gas Control Centre at the Head Office using the Supervisory Control and Data Acquisition System (SCADA). All compressor stations and meter stations will be designed for remote unattended operation. All systems within the stations will be designed to be failsafe and the facilities will continue to operate safely in the event of a communications failure. The Gas Control Centre will be manned 24 hours a day, seven days a week.

All communications systems required for the operations and maintenance of the pipeline will be provided by a leased satellite system with hub stations and ground stations, as required.

The project will prepare manuals covering maintenance procedures, standard operating practices and contingency procedures. Copies of these manuals will be maintained at all appropriate locations. All manuals will be subject to periodic review and will be revised to reflect changes in company policies, improvement in methods or equipment and as operating experience is gained.

The entire pipeline system will be patrolled routinely by fixed wing or rotary wing aircraft operating at low air speed and low altitude, and special attention will be given to slopes, river crossings and crossing approaches.

8.3 FIELD STAFFING AND EQUIPMENT

8.3.1 General

The organization proposed and the levels of staffing and equipment required for the operations and maintenance of the pipeline system take into account the general remoteness of most facilities and geographical constraints that may limit accessibility to the various facilities.

8.3.2 Staffing

The organization of the operations and maintenance group has been structured to centralize the management, and the technical, operations and maintenance personnel at locations considered to be the most appropriate in order to ensure the safe, reliable and efficient operation of the pipeline system.

The requirement to maintain a permanent staff of qualified, trained and competent personnel has also been taken into account. The field organization is divided into three Divisions, each of which will be headed by a Division Manager who will report to Head Office senior management personnel.

Division headquarters, which will be administrative offices, will be supervised by a Division Manager. The Head office will provide the Division Manager with specialized technical and

other support services that may be required, and this Manager will be assisted by the District staff at the field locations under his jurisdiction.

Each District will be supervised by a District Manager who will be supported by a crew of supervisors, technicians, utility or maintenance and station operations personnel. The Manager will be responsible for the day-to-day operations and maintenance functions within his area of responsibility, in accordance with the Operations Policies and Procedures of the Company.

Each District facility will consist of an administrative office, a mechanical and electric repair area and storage facilities.

8.3.2.1 <u>Northern Division</u>

Personnel will live in their own homes in the area of the Division and District Office locations. Transportation of personnel to the compressor stations and other facilities will normally be by ground transportation or, where required, will be provided by aircraft.

The rotation schedule in the Divisions will require double staffing for most job functions in the operation of the compressor stations. Accommodations will be provided at the compressor stations for operations and maintenance staff who will be rotated routinely.

8.3.2.2 <u>Central and Southern Divisions</u>

It is anticipated that Central and Southern Division employees will live in the general area of their headquarters office and they will work a normal work week, returning to their homes after working hours. Major maintenance or other special circumstances may require them to spend some time away from their headquarters. On these occasions company accommodation will be provided at the work site.

8.3.2.3 <u>Employee Training</u>

Whenever possible, the project will encourage residents of communities close to the pipeline to take advantage of full-time operations and maintenance employment opportunities. As part of this policy, the project will establish, in conjunction with the appropriate authorities, training programs for employees.

The ultimate aim of this policy will be to have most or all jobs filled by qualified residents who will be encouraged to live in the headquarters communities. As this aim is realized, the number of people required to work rotation schedules will be minimized. In addition to full-time employment opportunities, the employment of part-time workers or local contractors may be required during periods of high workloads. Where local people can fill the requirements, they will be given the first opportunities whenever possible.

8.3.3 Equipment

To support day-to-day operations and maintenance requirements of the pipeline system and in order to be able to deal with contingency situations, transportation and work equipment will be located at strategic locations along the pipeline.

In planning the locations and quantities of equipment, geographic constraints were considered, including the need to minimize right of way travel distances when carrying out pipeline repair work.

Besides the ground transportation and work equipment requirements, there will be a need for fixed wing and rotary wing aircraft. Typical uses for the aircraft will include the rotation of compressor station O&M personnel, the resupply of field facilities with consumables, spares and other materials, and the routine inspection flights along the pipeline and management travel.

The aircraft will be based permanently at headquarters locations as follows:

At Each Division:	1 large twin engine short takeoff and land (STOL) fixed wing aircraft
	1 light twin engine, short take-off and land (STOL) fixed wing aircraft
	1 large lift rotary wing aircraft
At Each District:	1 light lift rotary wing aircraft

8.3.4 Compressor Stations

The compressor stations will be remotely controlled from the control centre. However, in the more northerly stations the additional refrigeration and cooling facilities and equipment will require full-time maintenance staff. As well as performing routine maintenance, the staff would be capable of operating the equipment locally under emergency conditions.

8.3.5 Communication and Control

The entire pipeline system will be operated from the control centre. The control centre will be manned 24 hours a day throughout the year. Operators will be on duty at all times and there will be spare control systems in place which can be activated in the event of a failure of the main control system.

All relevant aspects of the operation at each delivery and receipt point and each compressor station will be monitored. Functions such as starting and stripping units, manipulating valves and controlling set points or pipeline temperatures will be performed by the control staff. The SCADA system will provide all necessary pipeline and station operations information to enable safe and efficient pipeline system operations.

If immediate attention is required at any station or remote facility, control personnel should advise the appropriate district supervisor who can make the appropriate arrangements.

8.3.5.1 <u>Communications Operations</u>

The communications system will consist of voice services from telephone and mobile radio, data services for the SCADA systems, e-mail and facsimile services. These services will be provided through above-ground lines, microwave, co-axial cable or satellite systems, or a combination of these will be utilized based on their economics and reliability for a certain area. For the more remote northerly areas, most of the preceding services may not be available and the most probable system will be leased satellite system with backup and the installation of "Hub" stations and a number of ground stations at remote sites. Maintenance of the satellite system would be the responsibility of the leasing company.

8.3.5.2 <u>SCADA and Control Operations</u>

The monitoring and control of all facets of pipeline operations will be provided by the SCADA system. Control will originate from the control centre's operator console which will allow the shift dispatcher to routinely start and stop compression units, open or close valves, change control, set points during normal operations and to initiate emergency shutdowns.

The operator console will present information such as system operating parameters, alarm status, valve positions and other data. This will be presented in the form of displays on screen.
At each station, controls to ensure safe operating conditions and unit protection would be provided independent of the control centre. Station controls will receive directives such as set point changes from the control centre to ensure that all stations are operating efficiently. If communication to the station is interrupted, the station will continue to operate safely at the last control directive received from the control centre. However, if necessary, control of the station can be shifted to the station panel and operated by the maintenance staff at the location if this is considered desirable.

8.3.6 Pipeline Dispatching

Dispatching in the pipeline is an integral part of pipeline operation. The natural gas pipeline will dispatch one common stream and one specification. It is only necessary to ensure that the customers are able to receive the required volume at any given time. Thus forecast of customer demands is an important part of designing the system to ensure adequate pipeline capacity. In some systems the high swings between winter and summer demand is moderated by installing storage capacity at or near the downstream end of the system. An economic evaluation of a particular system will determine whether it is preferable to install additional seasonal compression power for the system or install gas storage.

Another method of moderating the wide in seasonal demands is to have a percentage of the throughput in "interruptible" demand. At peak periods, the interruptible customers can be curtailed to the extent necessary to satisfy the demands of firm customers.

8.4 PIPELINE INSPECTION AND MAINTENANCE

8.4.1 Pipeline Patrol

The pipeline right of way will be patrolled on a regular basis by fixed wing or rotary wing aircraft at low airspeed and low altitude. The pipeline will be patrolled once a week, augmented by daily foot patrols in areas of high population density and heavy construction activities.

In the more northerly areas, spring breakup or as unusual conditions indicate the need, the patrol frequency will be increased. Land patrols, using all-terrain vehicles where possible, will be used to augment aerial patrols to investigate problem areas or for detailed surveys of the pipeline and the right of way.

8.4.2 Pipeline Maintenance

Pipeline maintenance will be carried out by district staff using equipment and materials from the facility most convenient in the work site. The repair work on the right of way will be carried out in an environmentally acceptable manner and only approved methods would be employed.

The inspection and maintenance activities in the initial years will be more intensive, to spot potential problems early and to initiate corrective solutions early to prevent any long term major shutdowns of the system. Thus the maintenance program can be broken down to short-term programs and long-term plans.

8.4.2.1 Pipeline Maintenance Short-Term Program

Immediately after construction, the right of way will be particularly susceptible to erosion and backfill settlement. For the first two to three years, extensive maintenance will be required.
The aerial patrols and land patrols should be increased in the early years to detect any potential problem areas soon to prevent serious problems particularly in the discontinuous permafrost area.

In the aerial surveillance the surface of the right of way will be observed for the development of local differential thaw settlement and significant changes in drainage patterns on permafrost terrain. Any such locations will be noted and identified for regular observation. Although differential thaw settlement of the pipe would not be expected to approach the design allowance until after a number of years of operation (where a warm pipeline is operated in the discontinuous permafrost area) there could be areas where deterioration could be more rapid due to local conditions. Thus, stepped up patrols for determination of surface differential thaw settlement would provide an early warning of potential pipe settlement.

Intelligent pigging runs should be utilized to detect more precisely the bending of the pipe due to thaw settlement. If significant differential thaw settlement develops, remedial action could include reducing the loads on the settling portion of the pipe or "softening" the thaw-stable support under the pipe, e.g. by water jetting.

All installations for observing thaw progression and potential high porewater pressures on slopes must be monitored regularly, in some cases on a monthly basis, at least in the early years. Visual observations by design engineers will be required in the late part of the thaw season - just before snow cover.

In the early years, river crossings are particularly susceptible to erosion in the riverbed and river banks. Therefore aerial inspection of crossings will be supplemented by underwater inspections at periodic intervals as well as when any abnormal condition is suspected. These inspections will check the ditch line for evidence of erosion and exposure of the pipe and will determine the river bottom condition. Photographic records should be maintained to monitor the charges in bank conditions. In permafrost areas the greatest concerns for pipeline integrity is the potential failure of slopes. Therefore attention will be given to all slopes in the permafrost areas as well as river crossing to ensure early detection of any potential problems.

In the early years subsidence of the ditch in ice-rich areas will be prevalent requiring the need to haul in suitable backfill material. Although the ditch subsidence in relatively flat areas will not pose a pipeline integrity problem, good maintenance practices and environmental concerns suggest that the pipeline be properly covered.

8.4.2.2 Pipeline - Long-Term Maintenance Plan

After the initial two to three years, conditions on the right of way should generally be more stable. The long-term maintenance plan for the pipeline will include a regular aerial patrol of the right of way, assisted by on-the-ground inspection in potentially troublesome areas. The exact frequency of the patrols will be dictated by the information and type of maintenance required in the initial years.

The patrols will be supplemented by regular intelligent pigging of the complete pipeline. Keeping a record of the findings will assist in detecting potentially troublesome areas at an early stage.

After a record has been developed in the early years of the pipeline, a long term maintenance program can be developed. Slopes monitoring must continue, with appropriate emphasis based on observed data in the first three years.

8.4.3 Maintenance of Stations and Other Facilities

Although all compression, measurement and other facilities would be designed for unattended operation, stations which rely on air access or are remote from housing facilities will be manned on a 24-hour basis. Operating staff will include a technician who would be responsible for day-to-day operating problems and operate the facilities from local control when necessary. Other staff would perform routine inspection and housing functions and would be responsible for ensuring that runways are kept clear and that aviation weather information is current and available.

General maintenance at stations will be the district responsibility and will be carried out in accordance with specific procedures. District technicians and traveling maintenance crews will be assisted, when required, by equipment vendor field service representatives or division headquarters and head office specialist personnel.

District maintenance staff would visit the various facilities to perform usual inspections, schedule overhauls, trouble-shooting, and housekeeping tasks. Meter stations and other such facilities would be visited on a less frequent basis for visual inspection and housekeeping duties.

Major maintenance of rotating equipment and ancillary facilities will be undertaken by specialized personnel from division headquarters. The extent of major repairs and maintenance required would dictate whether the work would be performed on site or transported to division headquarters to affect repair procedures.

8.5 LEAK MONITORING AND RESPONSE

Aerial patrols and ground reconnaissance will detect major leaks and ruptures. However, small leaks may go undiscovered for a considerable period of time, without some for of measurement and reconciliation of supplies and delivery volumes. For sizable leaks, a shutdown procedure would be activated automatically at intermediate valve locations. However, for small leaks, it is difficult to institute a shutdown system. Experience with the system operation and maintenance of accurate line balances will greatly assist in detecting smaller leaks.

8.6 CONTINGENCY PLANS

The project will develop contingency plans to ensure a rapid and acceptable response to any unexpected occurrence on the pipeline system. These plans will be designed to ensure public safety, minimize environmental damage and maintain or restore operations of the pipeline system. An adequate inventory of pretested pipe, pipe fittings, valves, rotating machinery parts and other materials should be maintained to support these plans.

8.6.1 Pipeline Contingency Plans

Detailed contingency plans will be developed for each pipeline section to take into account all available design data, as-built construction and environmental data. The contingency manuals will include information such as best methods of access, communications procedures and availability of local labour and equipment.

Two methods of pipeline repairs are available. The first method would be used when right of way travel is possible and would involve the removal and replacement of the affected section of pipeline with full size pipe in the conventional manner. In permafrost areas this may only be possible in winter where ice-rich soils are present. It may be possible to install permanent repairs during the summer in certain areas provided thaw stable soils exist and overland access is possible.

The second method would be used when right of way travel by heavy equipment is not possible at that time of year. The repair would be carried out using stopple equipment and smaller diameter pipe to construct a temporary bypass around the defective section of

mainline. All repair pipe, fittings and work equipment would need to be transported to the work site by special all-terrain carriers or heavy lift helicopters.

The temporary bypass repairs could be removed and replaced by permanent full size pipe in the conventional manner during the following winter.

The responsibility for carrying out contingency repairs will rest with the appropriate district manager who would be notified by pipeline control or on-site personnel. All other appropriate authorities and head office personnel would need to be informed of the situation at the same time.

Mainline valves will be equipped with low pressure detection devices. In the event of a line break, the upstream and downstream valves would close automatically to isolate the sections. Company personnel, equipment and materials would be dispatched to the work site to commence repairs as soon as safe conditions exist.

8.6.2 Station Contingency Plans

Procedures for dealing with station contingencies will be included in the contingency manuals and initiated by the district manager upon being notified of the emergency situation. Each station will be protected by hazardous atmosphere and fire detection systems and each location should be equipped with appropriate fire-extinguishing systems. An initiation of the detection systems would result in the facility being shut down safely and isolation from the mainline.

Sufficient spare parts and equipment should be available within each district to carry out any anticipated repairs. For major repairs assistance would be required from division headquarters and the main maintenance base.

8.6.3 Fires

All operations personnel will receive training in fire control procedures and use of extinguishing equipment. Regular drills will be held to ensure that all personnel are up-to-date with the latest techniques. All pipeline facilities and mobile equipment will be provided with fire-extinguishing equipment.

9.0 CONCLUSIONS

9.1 FEASIBILITY OF PROPOSED PIPELINE

The author has presented the need for Japan to secure adequate sources of piped natural gas (PNG) for the early 21st century. PNG is preferred, to replace Japan's dependance on environmentally unacceptable fossil fuels and nuclear energy, and to provide a potentially cheaper energy source compared to LNG. While Sakhalin Island gas resources will provide the first opportunity for Japan to import significant PNG, the next likely source for PNG is expected to be East Siberia. Hence, the author chose to consider the feasibility of a natural gas pipeline from East Siberia, through Mongolia to the Chinese coast near Tianjin.

Any pipeline starting in East Siberia will have to be constructed in permafrost terrain. Considering pipelines in permafrost, the author was only aware of the very expensive Alyeska pipeline and the many serious problems with the Russian pipelines. The author therefore set out to learn how to design, construct and operate an economic and reliable pipeline in permafrost.

In Russia there are many pipelines in permafrost, however, the quality of the pipe materials, equipment and the overall quality of construction is generally poor. The result has been numerous pipe ruptures. In North America, there have been considerable research and design studies conducted over the last 25 years, though there is less experience with actual construction. Referring to this North American expertise, economic and reliable designs have been developed for the proposed pipeline.

The thesis has presented information on the distribution and characteristics of permafrost, as a background to the presentation of the many design issues for pipeline projects in permafrost. Several pipeline design alternatives have been discussed and the preferred, less expensive, buried pipeline design approach is presented in more detail.

The author has pointed out the following items regarding the proposed pipeline.

1. It is shown that the best route runs from the Irkutsk area through Mongolia and down to Tianjin on the east coast of China, with a total length of about 2,900 km.

2. It will be necessary to conduct specific engineering studies since the pipeline crosses permafrost regions and also many rivers and mountain ranges.

3. It will also be necessary to establish an economical design and construction plan in order to compete with the existing LNG costs.

4. It will also be important to resolve significant geopolitical issues since the pipeline passes through Russia, Mongolia and China.

Furthermore the author has recommended the following for the design, construction and operation of the gas pipeline from East Siberia:

5. The plastic design criteria is a rational design method for buried pipelines in permafrost. The ASME B31.8 design code can be used for this purpose.

6. The proposed pipeline design can withstand a large amount of differential thaw settlement. For example, a pipe of API 5L X-70, with 22.2 mm wall thickness, can withstand up to 1.1 m of differential settlement.

7. Winter construction is recommended to minimize the environmental disturbance in the permafrost terrain. Jumbo wheel ditchers will be effective for trenching in frozen ground.

8. The installation of ground-cooling thermosyphons will be effective to maintain the stability of the pile foundations for the compressor stations in permafrost.

9. The proposed buried pipeline can be constructed in 3 years.

10. Regular patrols of the pipeline should be focused on the detection of any potential drainage or erosion problems and significant thaw settlement that may develop.

11. The latest "intelligent" pigging devices will be effective for the advance detection of any significant pipe curvature development due to differential pipe settlement.

12. Insulation must be applied to the slopes in ice-rich permafrost to protect against slope failure by thawing . The use of wood chip insulation will be effective for the slope stability.

Based on these design and construction approaches, the capital and operating costs for the proposed East Siberian pipeline project have been estimated. The author has compared these costs with published costs for major pipelines.

The author is able to conclude from this feasibility study that the proposed 2875 km long, 1422 mm diameter natural gas pipeline can be constructed for a capital expenditure of US$ 6,883.4 million. Taking into account the total capital costs and the operations and maintenance costs, the expected average cost of transport for the gas would be in the order of US$ 1.29/ mcf over a 20 year period, based on a 15% internal rate of return and a 10% interest rate on debt. As such the project is considered economically viable.

The author concludes that this project is very feasible and recommends that it be given serious consideration in Japan's energy planning for the early 21st century.

9.2 FURTHER RESEARCH REQUIRED

There are several matters related to the overall project, that were beyond the scope of this study, but need to be stated as required study items for a more complete assessment of the feasibility of the pipeline project:

- Environmental and socio-economic impact assessments, which will form the basis for much of the regulatory review and permitting process in Russia, Mongolia and China,
- Determine the precise regulatory review and permitting process in each country,

- Confirmation of gas supply and build up scenarios,
- Confirmation of gas markets - initial and prospective,
- Research the project financing options,
- Establish fiscal terms to be applied to the project,
- Establish the multi-national ownership/equity basis for the project,
- Establish the multi-national operations and control structure for the project

For research more specifically related to this thesis, the author has recognized several items, listed below. Some of these items will require a sense of direction from the main project proponents and the respective governments.

1. Select more optimum routing to reduce the length, and therefore the cost of the pipeline,
2. Determine the available, existing data bases regarding terrain and environmental conditions, at local institutes all along the route,
3. Determine the capabilities of the local contractors regarding availability and costs for equipment and skilled labour.
4. Assess all logistical capabilities, constraints and rates.
5. Assess quality and capabilities of local supply for materials.
6. Assess quality and capacity of local compression unit manufactures.
7. Pipeline system optimization with respect to pipe diameter and pressure, based on updated throughput forecasts.
8. Assess merits of automation of compressor stations; fully automated compressor stations can be unmanned except for scheduled or emergency maintenance.
9. Hold technical meetings with Russian (and Mongolian) pipeline research and design institutes to negotiate acceptance of ASME B31.8, and in particular, the plastic strain based design criterion for thaw settlement design,
10. Assess benefits of internal pipe coating.

The above items are presented in the order of the design process rather than in order of priorities. In fact, the author considers that Item 9 is the most critical in terms of project viability and recommends these meetings should be initiated as early as possible. In addition, Items 1 to 4, which address pipeline length, local capabilities and productivity, as well as logistical constraints, could also result in significant economic impacts.

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APPENDIX A

Gas Pipelines in Permafrost - Stress/Strain Design Criteria

1. STRESS CRITERIA

The gas pipeline will probably be subject to the requirements of the ASME B31.8 - Gas Transmission and Distribution Piping Systems Code, as is typical for many international projects. The stress criteria of this code are as follows:

- internal pressure stress shall not exceed 0.72 SMYS
- equivalent tensile stress for restrained lines due to combined effects of thermal differential and fluid pressure shall not exceed 0.9 SMYS
- the expansion stress range for unrestrained lines due to combined effects of thermal differential and fluid pressure shall not exceed 0.72 SMYS
- the sum of longitudinal stresses due to pressure, weight, and other sustained external loadings shall not exceed 0.54 SMYS
- the sum of longitudinal stresses due to pressure, weight, other sustained external loadings PLUS occasional primary loads shall not exceed 0.80 SMYS.

These criteria place limits on the typical primary stresses encountered for pipelines and on typical secondary stresses (i.e. due to thermal load) where fatigue is of concern. The "sustained external loadings" are understood to be of a primary nature, i.e. are not displacement limited.

2. STRAIN CRITERIA

For the displacement-controlled loads potentially encountered on this project it is proposed to utilize limit state design with strain based criteria. The displacement-controlled loads potentially encountered on this projects are thaw settlement, frost-heave, seismic loads on buried pipe, thermal expansion at buried bends, roping-in curvature and ditch-bottom curvature.

This is a rigorous and rational approach, since it addresses the potential modes of pipeline damage and failure. The damage mode being guarded against, resulting from the large axial compressive strains, is local buckling which could significantly affect the pipeline operation, i.e. reduce gas flow and prevent pigging. The ultimate failure mode being guarded against is tensile rupture which could occur due to various excessive tensile strains, including those in the area of a local compressive buckle in the pipe wall.

The ASME B31.8 Code for Gas Transmission and Distribution Piping Systems does address strain-based design and is an appropriate source. It states, in paragraph A842.23 entitled Alternate Design For Strain,:

"In situations where the pipeline experiences a predictable noncyclic displacement of its support (e.g....) or pipe sag before support contact, the longitudinal and combined stress limits need not be used as a criteria for safety against yielding, so long as the consequences of yielding are not detrimental to the integrity of the pipeline. The permissible maximum longitudinal strain depends....".

In order to determine appropriate design criteria, as allowed per the codes discussed above, other design codes and technical documents provide assistance. The ASME Boiler and Pressure Vessel Code, Section VIII, Division 2, provides guidelines for limit state analysis; this code is typically more rigorous than the related piping codes.

2.1 Tensile Strain

The tensile strain limit typically does not exceed 0.5% in the oil and gas pipeline industry; a design factor is usually applied to this limit resulting in typical allowable design longitudinal strain values of 0.35% to 0.4%. These allowables are applicable to standard weld procedures and inspection. It should be noted that apart from the welds, most pipeline steel manufactured today can withstand tensile strains of much greater than 1.0%.

With adequate fracture mechanics analysis and/or physical testing to establish acceptable circumferential weld flaws, and then with proper quality control and weld inspection procedures, the tensile strain limits could be increased. This should be considered if it is determined to be beneficial to the project. (e.g.; if it would have a reasonable probability of preventing costly repairs due to soils displacements).

It is proposed to utilize allowable design longitudinal strain (tensile) limited similar to the following examples, based on inelastic analysis:

- 0.55% strain for combined bending and membrane strains due to static loads of pressure, thermal differential and displacement-controlled loads.
- 0.63% strain for combined bending and membrane strains due to static plus dynamic loads of pressure, thermal differential and displacement-controlled loads including design probable earthquake.
- 0.70% strain for combined bending and membrane strains due to static plus dynamic loads of pressure, thermal differential and displacement-controlled loads including design maximum earthquake.

2.2 Compressive Strain

For initially straight pipe, it is proposed to utilize allowable design longitudinal strain (compressive) limits similar to the following examples for pipe of 56" OD x 0.875" WT x Gr 65 ksi, based on inelastic analysis:

- -0.55% strain for combined bending and membrane strains due to static loads of pressure, thermal differential and other displacement-controlled loads.
- -0.66% strain for combined bending and membrane strains due to static plus dynamic loads of pressure, thermal differential and other displacement-controlled loads including design probable earthquake.

- -0.78% strain for combined bending and membrane strains due to static plus dynamic loads of pressure, thermal differential and other displacement-controlled loads including design maximum earthquake.

Axial compressive strain (E c) limits are incorporated in order to prevent local buckling of the pipe. This limit is a function of the pipe diameter and wall thickness. The allowable compressive axial strain will be less for initially bent pipe, compared to initially straight pipe.

Internal pressure increases the buckling strain by the addition of the percentage strain value as determined by the following formula:

$$+300\ [Pi \times D/\ (2 \times 5 \times E)] \times F \times 100\%$$

where:
- Pi = internal pressure
- D = outside diameter
- t = wall thickness
- E = Young's modulus (elastic)

The design probable earthquake (DPE) may be determined as that associated with a return period for the design of 50 years; the design maximum earthquake (DME) may be determined as that associated with a return period for the design of 100 years.

The rationale for the suggestion of incorporating an allowable axial compressive strain of 0.55% for the 56" pipeline is as follows. Langner (1984) presents a critical compressive strain formulation of 0.5 (t/D), i.e. = 0.781%; it is proposed to apply a design factor of 0.7 to this value.

Lagner's formulation was shown to be conservative in the paper by personnel of SSD Engineering Consultants Inc. And YPC entitled "Arctic Pipeline Limit States for Secondary Loadings". The paper presented at ASME's 1992 OAME conference, by Colquhoun, Price, Zimmerman and Smith entitled "Development of a Limit States Guideline for the Pipeline Industry" states "wrinkling is a gradual process... the point at which wrinkling is first perceived is likely less important than the point at which rapid wrinkle growth occurs. This can be at a strain many times that of wrinkle initiation... in one of Bouwkamp's tests, buckle initial was reported to have occurred at a strain of 0.58%, yet large wrinkle deformations were only calculated to have occurred after a strain of 2.5%.

3. OVALIZATION CRITERIA

The ovalization deformation is calculated as:

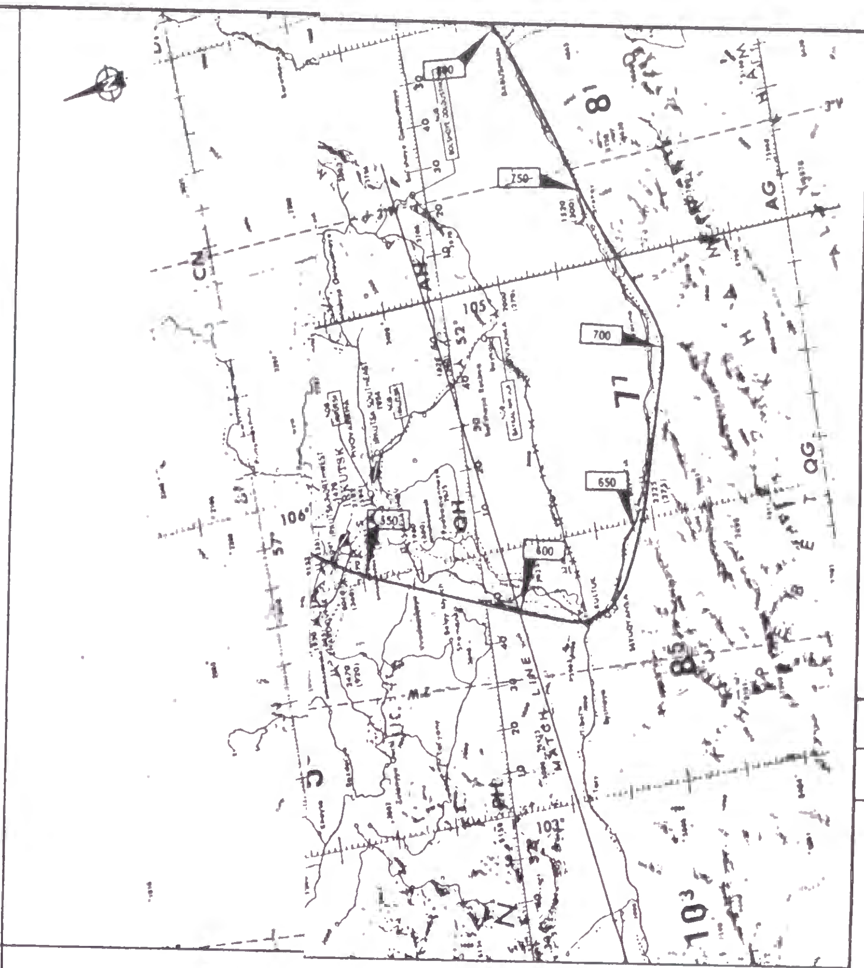
$$2\ [\ (Dmax - Dmin)/(Dmax + Dmin)].$$

A suggested limit is 0-.15 (10.0%) for construction loads (where Smax and Dmin are the maximum and minimum pipe outside diameters).

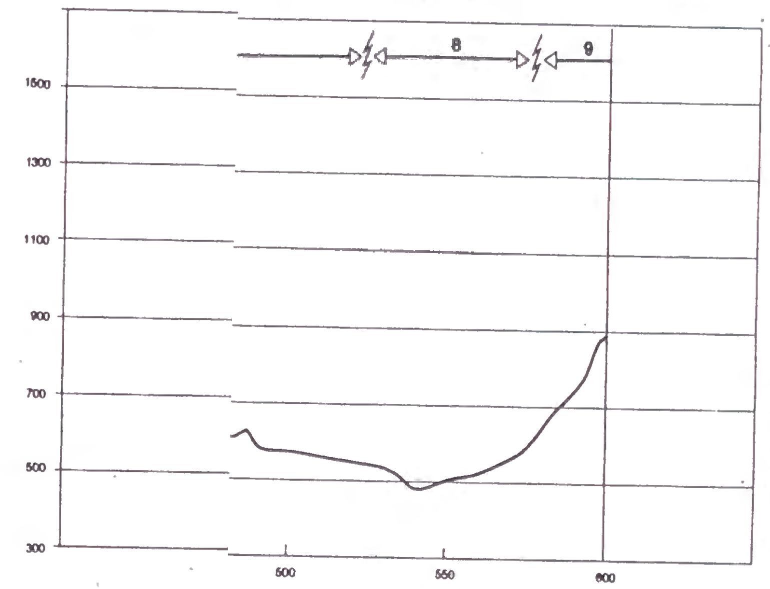
APPENDIX B

Pipeline Alignment Sheets

PLAN



ELEVATION PROFILE



PIPE DATA

← TRAVEL

SPREAD 4

480

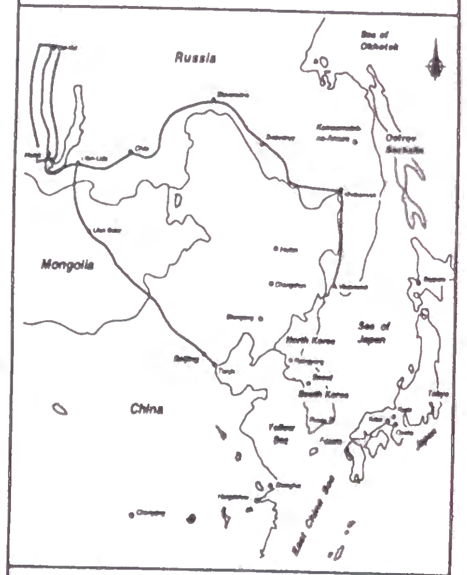
KP540 ANGARA RIVER

KP 585 IRKUT RIVER

TERRAIN AND CONSTRUCTION DATA

VEGETATION/LAND USE	FIELD		FOREST	
SURFICIAL GEOLOGY			SOFT ROCK	
PERMAFROST/ICE CONT.	MODERATE TO HIGH	0% / 10 - 15% MODERATE TO HIGH	15 - 20% LOW	
ROCK GRADE			RIPPABLE	
ROCK DITCH			TRENCHABLE	
CLEARING	NO CLEARING			MACHINE CLEARING
BUOYANCY CONTROL	WEIGHTING		NONE	

INDEX MAP



LEGEND:

PIPELINE ROUTE

CONSTRUCTION STAGING AREA

COMPRESSOR/PUMP STATION

SEISMIC ZONES

SPREAD BOUNDARIES

CLIENT:

NIPPON STEEL CORPORATION

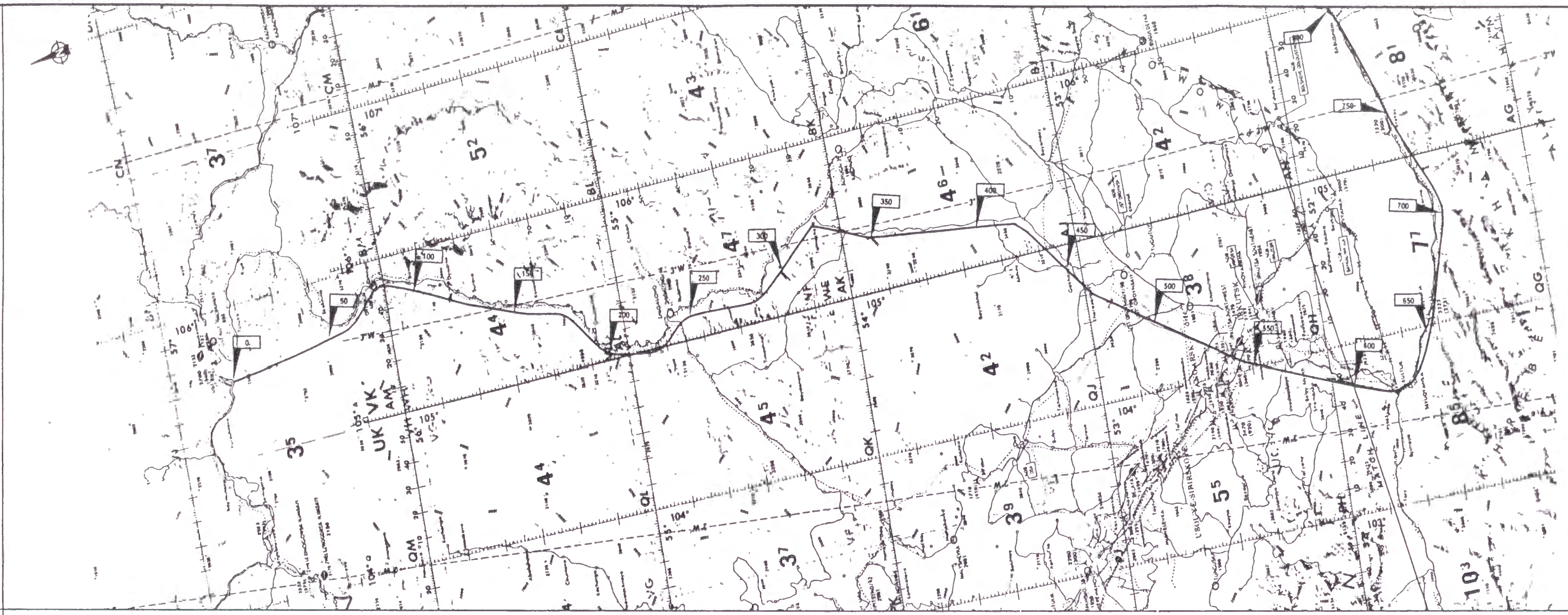
TOKYO JAPAN

EAST SIBERIA AND FAR EAST PIPELINES

PRE-FEASIBILITY STUDY

DRAFTED:		DATE:	
CHECKED:		JOB NO.:	
SCALE:	AS SHOWN	SHEET NO.:	CG 1 of 5

PLAN

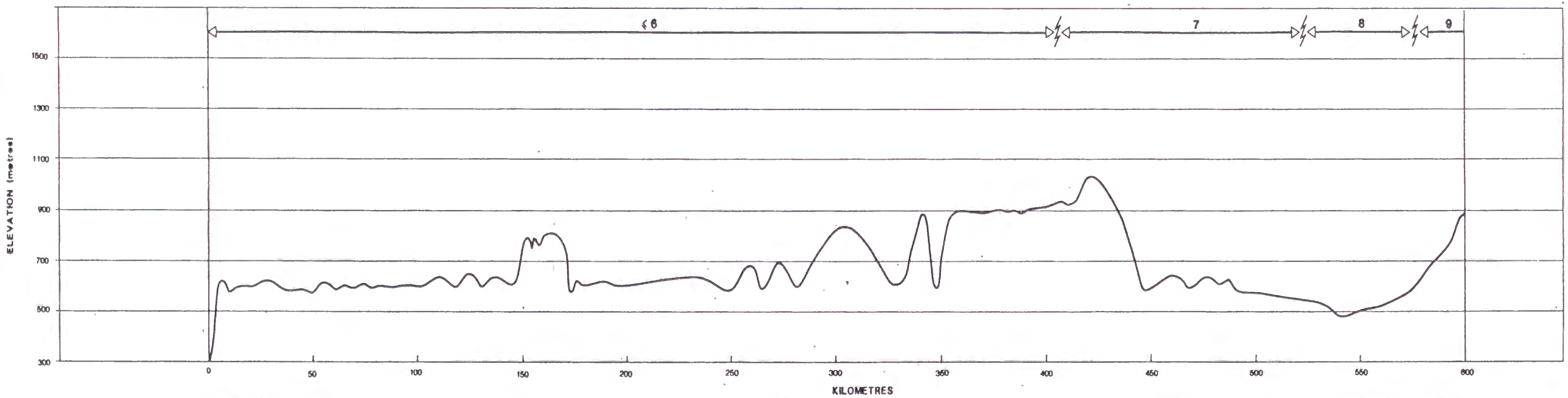


INDEX MAP

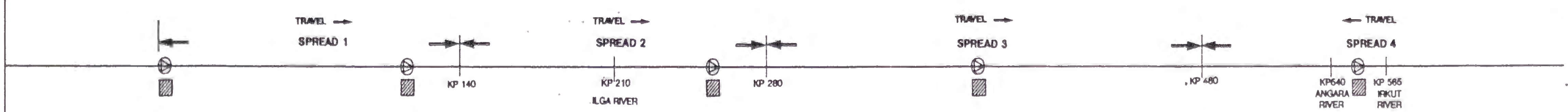
LEGEND:

- PIPELINE ROUTE
- CONSTRUCTION STAGING AREA
- COMPRESSOR/PUMP STATION
- SEISMIC ZONES
- SPREAD BOUNDARIES

ELEVATION PROFILE



PIPE DATA



TERRAIN AND CONSTRUCTION DATA

VEGETATION/LAND USE	FOREST	FOREST PROTECTED	FOREST	FOREST PROTECTED	FIELD	FOREST	FIELD	FOREST		
SURFICIAL GEOLOGY	SOIL	SOFT ROCK	SOIL	SOFT ROCK	SOIL	SOFT ROCK	SOIL	SOFT ROCK		
PERMAFROST/ICE CONT.	15-20% /MOD	30 - 35% / LOW	15 - 20% / MODERATE	30 - 35% / LOW	15 - 20% / MODERATE	30 - 35% / LOW	10 - 15% MODERATE TO HIGH	0%	10 - 15% MODERATE TO HIGH	15 - 20% LOW
ROCK GRADE	NONE	RIPPABLE ROCK	NONE	RIPPABLE	NONE	RIPPABLE	NONE	RIPPABLE		
ROCK DITCH	NONE	TRENCHABLE WITHOUT BLASTING	NONE	TRENCHABLE	NONE	TRENCHABLE	NONE	TRENCHABLE		
CLEARING	MACHINE CLEARING	MACHINE CLEARING	NONE	MACHINE CLEARING	NO CLEARING	MACHINE CLEARING				
BUOYANCY. CONTROL	NONE	WEIGHTING	NONE							

CLIENT:

NIPPON STEEL CORPORATION

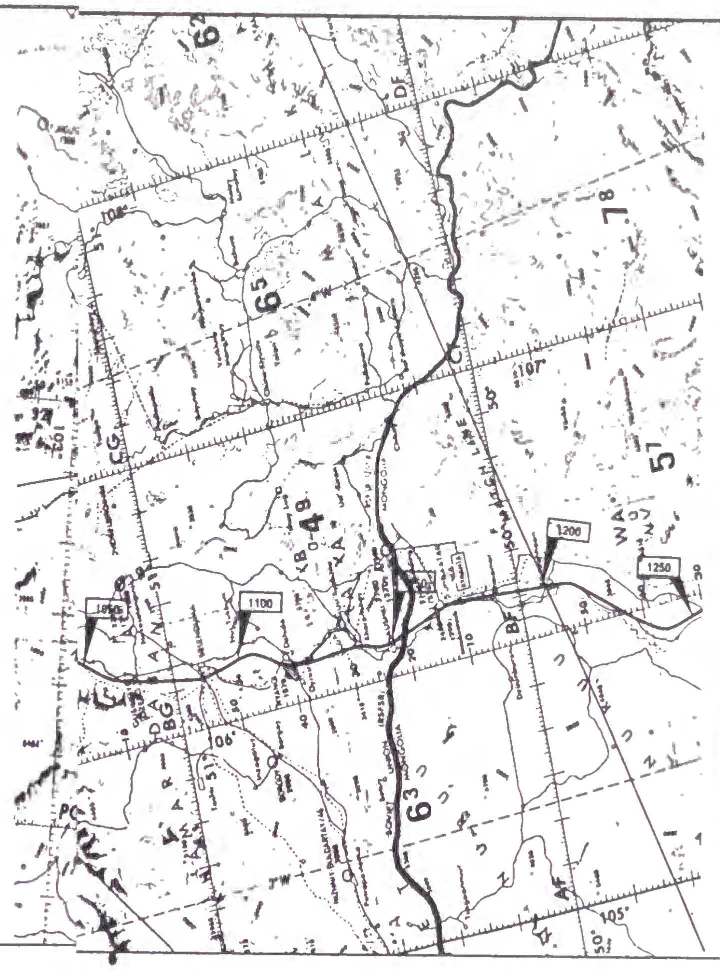
TOKYO JAPAN

EAST SIBERIA AND FAR EAST PIPELINES

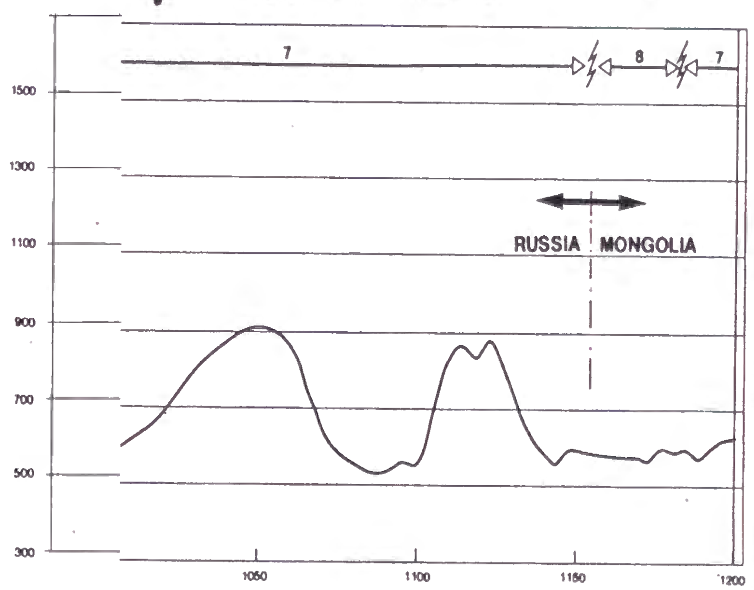
PRE-FEASIBILITY STUDY

DRAFTED:	DATE:
CHECKED:	JOB NO.:
SCALE: AS SHOWN	SHEET NO.: CG 1 of 5

PLAN



ELEVATION PROFILE



PIPE DATA

TRAVEL
SPREAD 7
TRAVEL
SPREAD 8
KP 1140
KP 1152
SELENGA RIVER

TERRAIN AND CONSTRUCTION DATA				
VEGETATION/LAND USE	FIELD	FOREST PROTECTED	FIELD	
SURFICIAL GEOLOGY	SOIL	HARD ROCK	SOFT ROCK	SOIL
PERMAFROST/ICE CONT.	/ MODERATE TO HIGH	10% /LOW		10% /LOW TO MODERATE
ROCK GRADE	ONE	BLASTING	RIPPABLE	NONE
ROCK DITCH	ONE	BLASTING	TRECHABLE	NONE
CLEARING	NONE	MACHINE CLEARING	NONE	
BUOYANCY CONTROL	EIGHTING	NONE		

INDEX MAP

LEGEND:

PIPELINE ROUTE
CONSTRUCTION STAGING AREA
COMPRESSOR/PUMP STATION
SEISMIC ZONES
SPREAD BOUNDARIES

CLIENT:

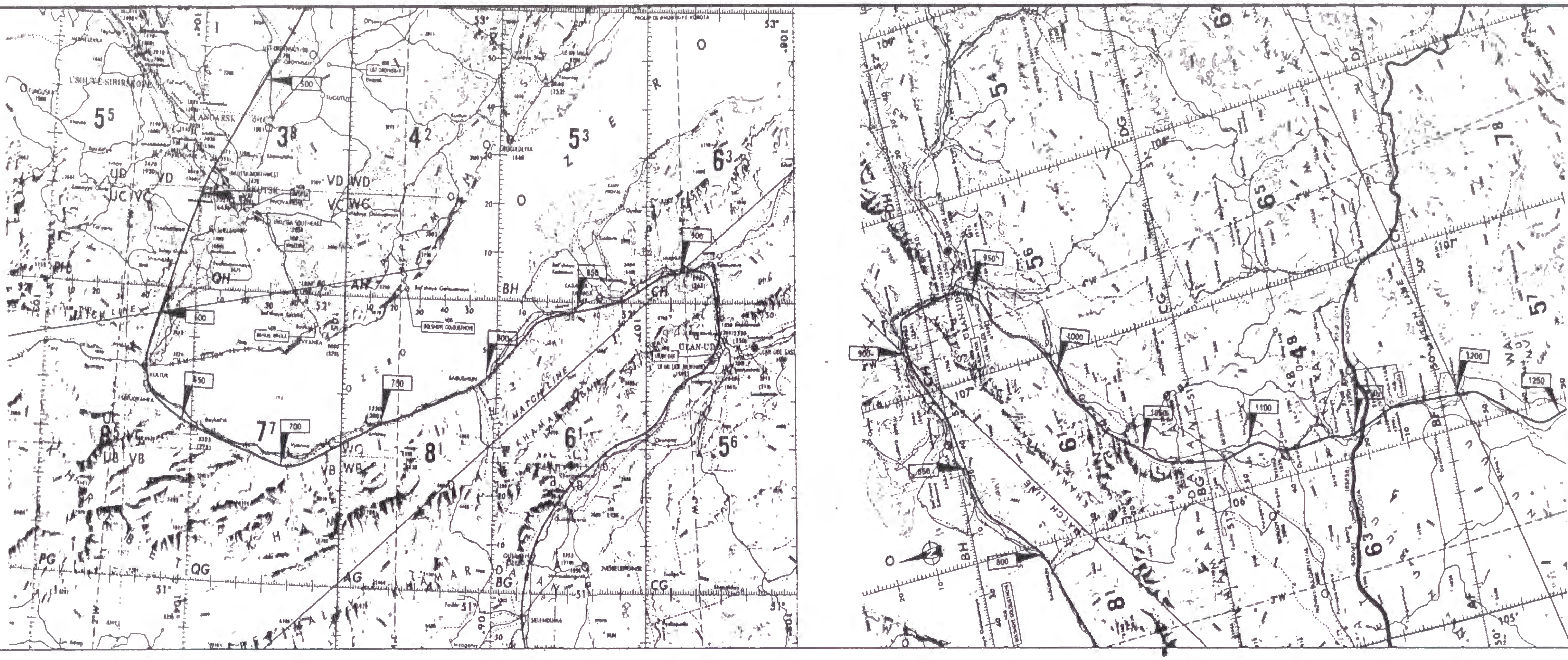
NIPPON STEEL CORPORATION
TOKYO JAPAN

EAST SIBERIA AND FAR EAST PIPELINES

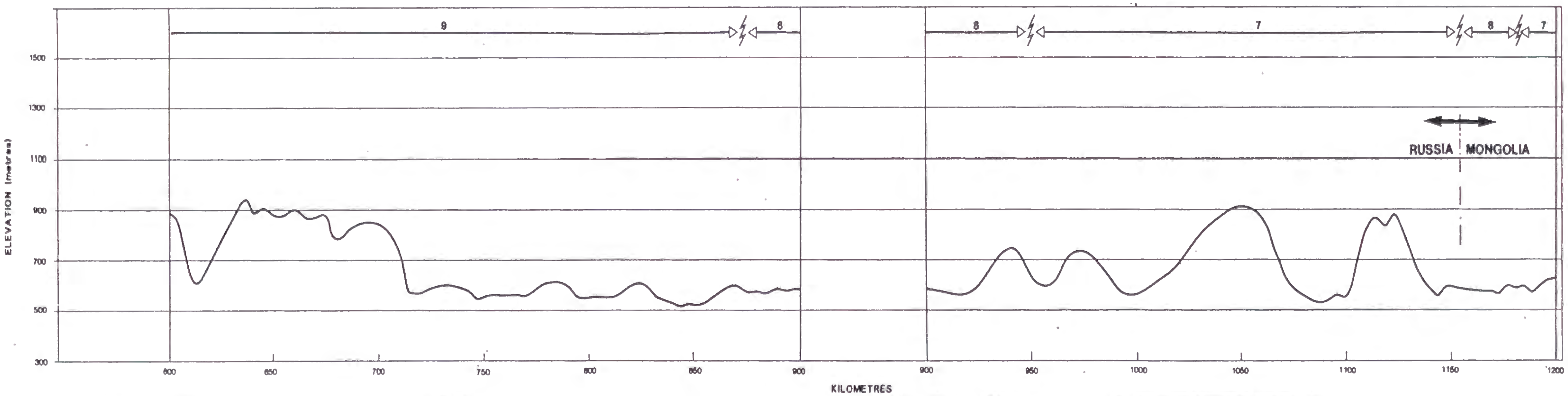
PRE-FEASIBILITY STUDY

DRAFTED:		DATE:	
CHECKED:		JOB NO.:	
SCALE:	AS SHOWN	SHEET NO.:	CG 2 of 5

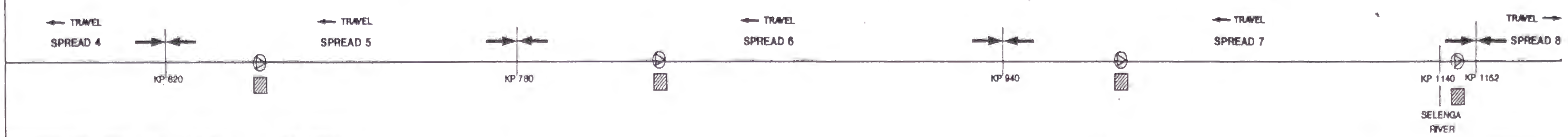
PLAN



ELEVATION PROFILE



PIPE DATA



TERRAIN AND CONSTRUCTION DATA

VEGETATION/LAND USE	FOREST PROTECTED	FOREST PROTECTED	FIELD	FOREST PROTECTED	FIELD	FOREST PROTECTED	FIELD		
SURFICIAL GEOLOGY	SOFT ROCK	HARD ROCK	SOIL	HARD ROCK	SOIL	HARD ROCK	SOFT ROCK	SOIL	
PERMAFROST/ICE CONT.	10 - 15% /LOW	10% /LOW	10% / MODERATE TO HIGH	20 - 25% / MODERATE	20 - 25% / MODERATE →	20 - 25% / LOW	10% / MODERATE TO HIGH	10% /LOW	10% /LOW TO MODERATE
ROCK GRADE	RIPPABLE	BLASTING	NONE	BLASTING	NONE	BLASTING	RIPPABLE	NONE	
ROCK DITCH	TRENCHABLE	BLASTING	NONE	BLASTING	NONE	BLASTING	TRECHABLE	NONE	
CLEARING	MACHINE CLEARING	MACHINE CLEARING	NONE	MACHINE CLEARING	NONE	MACHINE CLEARING	NONE		
BUOYANCY CONTROL	NONE	WEIGHTING	NONE	NONE	WEIGHTING	NONE			

INDEX MAP

LEGEND:

PIPELINE ROUTE

CONSTRUCTION STAGING AREA

COMPRESSOR/PUMP STATION

SEISMIC ZONES

SPREAD BOUNDARIES

CLIENT:

NIPPON STEEL CORPORATION

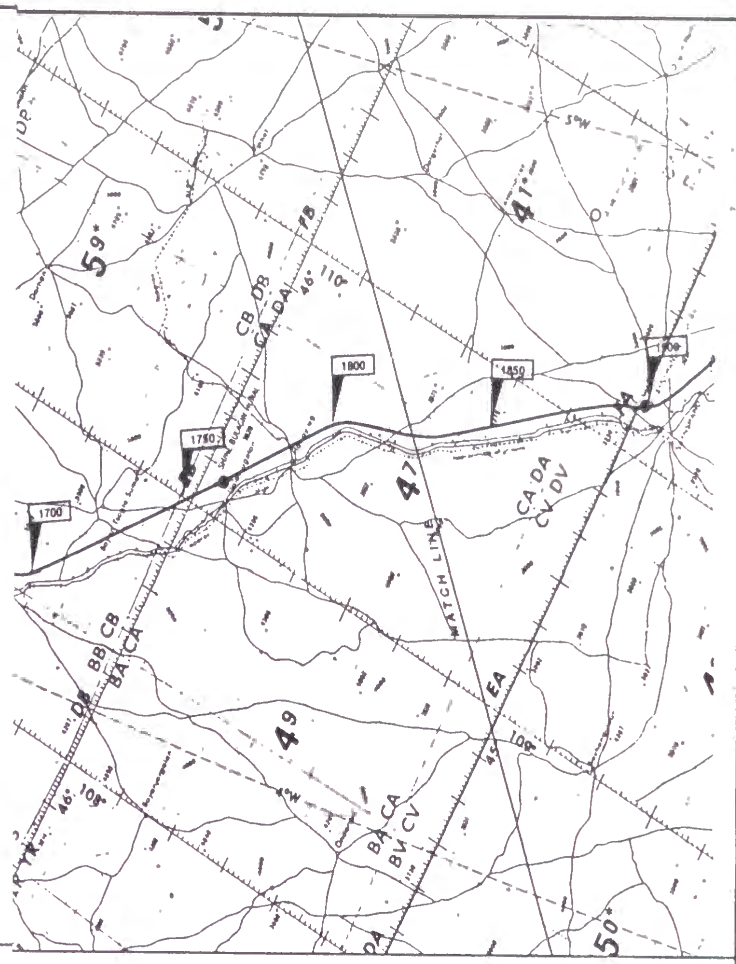
TOKYO JAPAN

EAST SIBERIA AND FAR EAST PIPELINES

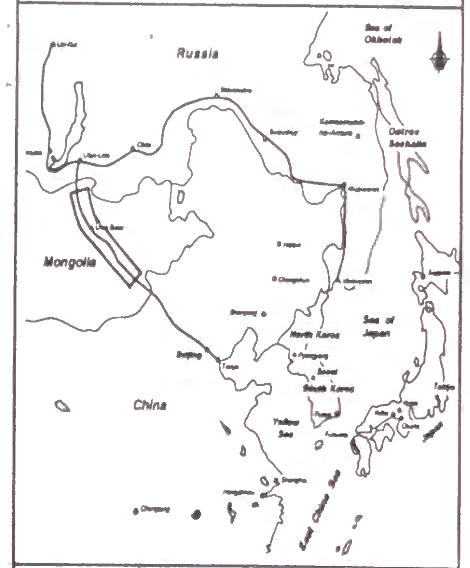
PRE-FEASIBILITY STUDY

DRAFTED:	DATE:
CHECKED:	JOB NO.:
SCALE: AS SHOWN	SHEET NO.: CG 2 of 5

PLAN



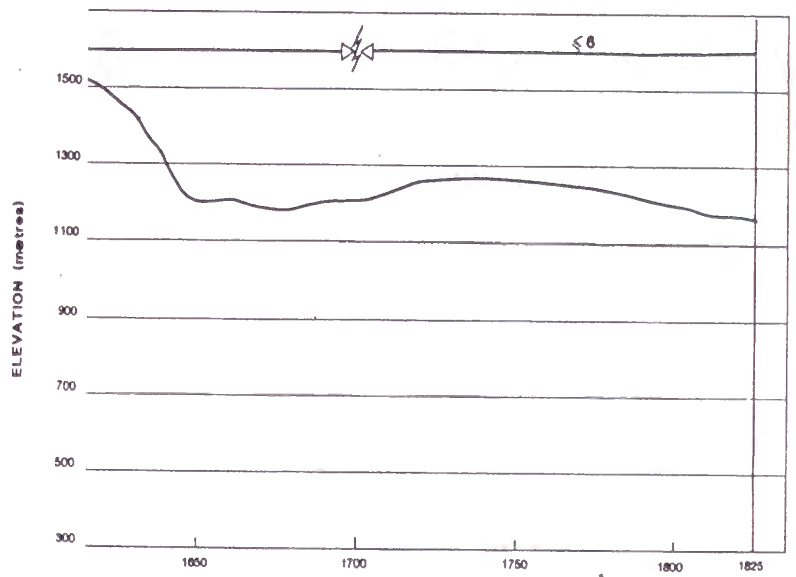
INDEX MAP



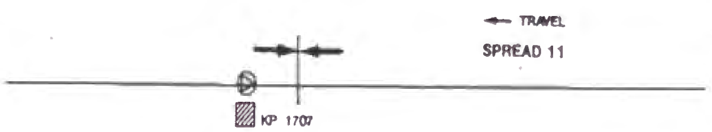
LEGEND:

- PIPELINE ROUTE
- CONSTRUCTION STAGING AREA
- COMPRESSOR/PUMP STATION
- SEISMIC ZONES
- SPREAD BOUNDARIES

ELEVATION PROFILE



PIPE DATA



TERRAIN AND CONSTRUCTION DATA

VEGETATION/LAND USE					
SURFICIAL GEOLOGY	SOFT ROCK	SOIL	SOFT ROCK	SOIL	SOFT ROCK
PERMAFROST/ICE CONT	NONE				
ROCK GRADE	RIPPABLE	NONE	RIPPABLE	NONE	RIPPABLE
ROCK DITCH	TRENCHABLE	NONE	TRENCHABLE	NONE	TRENCHABLE
CLEARING					
BUOYANCY CONTROL	NONE				

CLIENT:

NIPPON STEEL CORPORATION

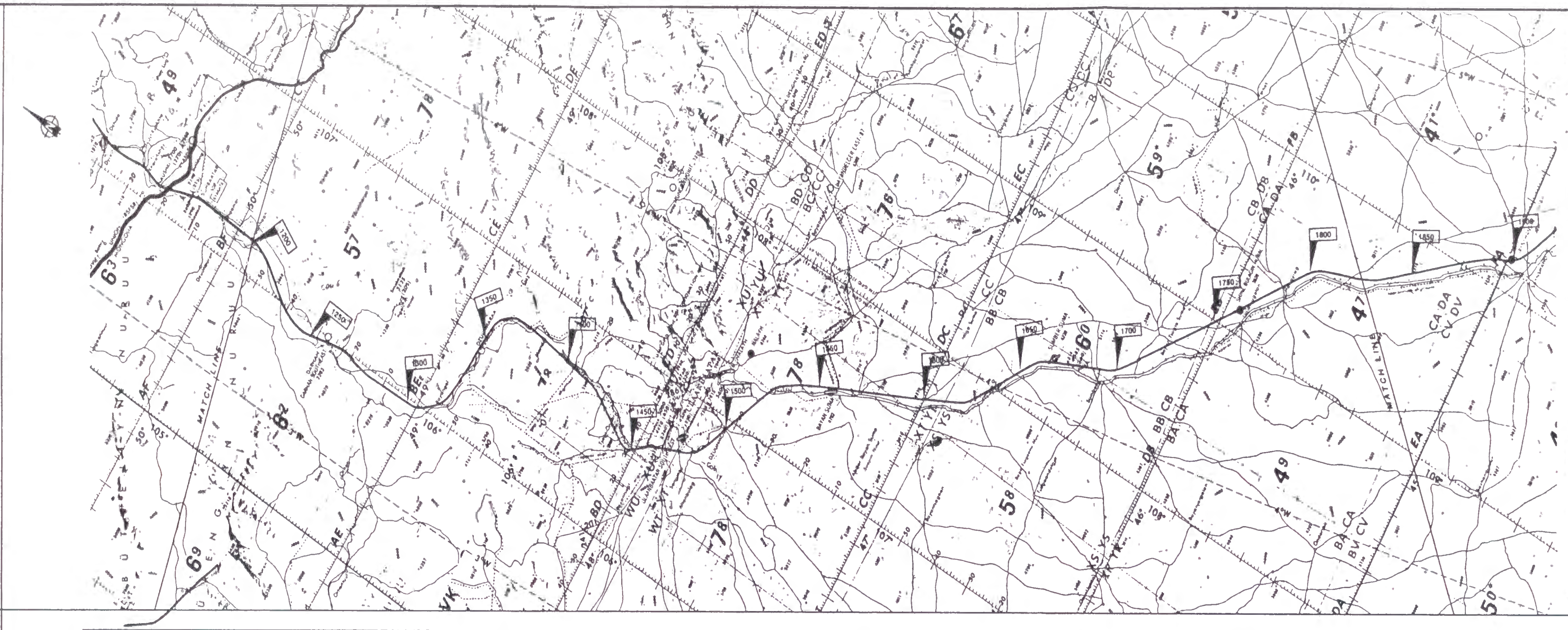
TOKYO JAPAN

EAST SIBERIA AND FAR EAST PIPELINES

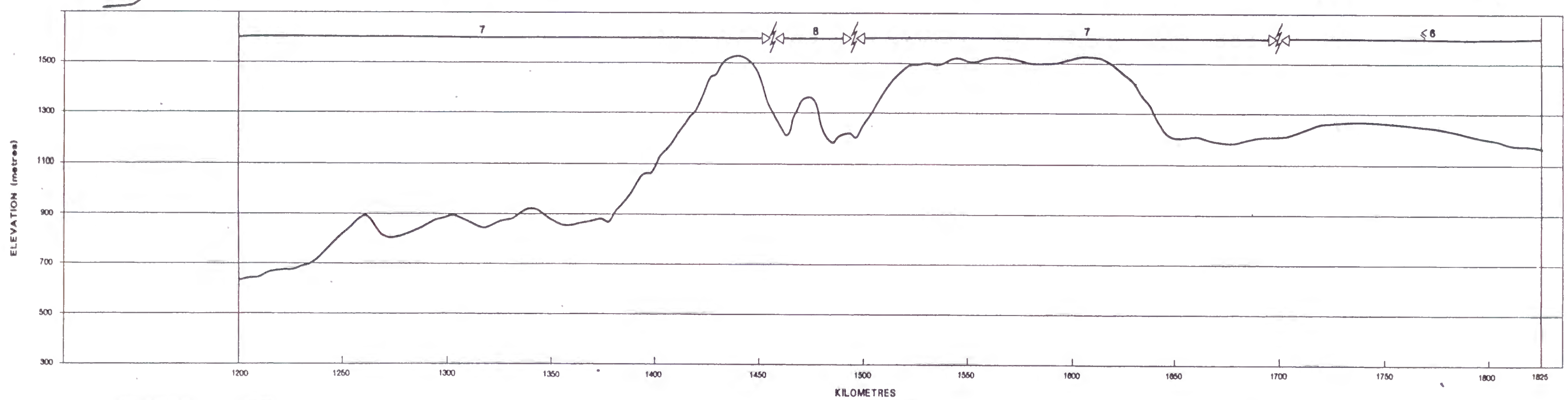
PRE-FEASIBILITY STUDY

DRAFTED:	DATE:
CHECKED:	JOB NO.:
SCALE: AS SHOWN	SHEET NO.: CG 3 of 5

PLAN



ELEVATION PROFILE

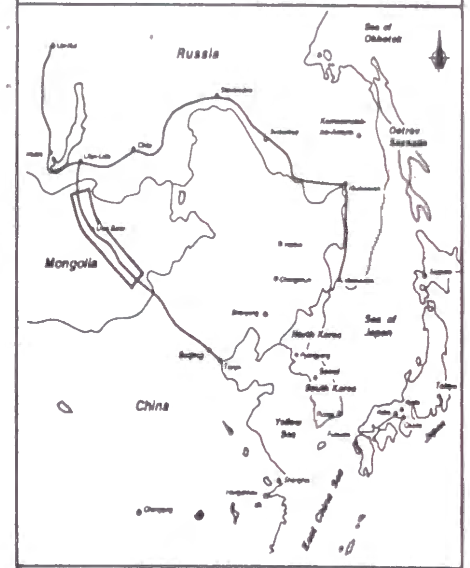


PIPE DATA

TERRAIN AND CONSTRUCTION DATA

VEGETATION/LAND USE	FIELD														
SURFICIAL GEOLOGY	SOIL		SOFT ROCK	SHALLOW HARD ROCK	SOFT ROCK	SOIL	SOFT ROCK	SOIL	SOFT ROCK	SOIL	SOFT ROCK	SOIL	SOFT ROCK	SOIL	SOFT ROCK
PERMAFROST/ICE CONT.	10% /LOW TO MODERATE	20-40% / LOW	20-40% / LOW MODERATE			5% HIGH	10% /LOW	5% HIGH	10% /LOW	5% HIGH	NONE				
ROCK GRADE	NONE		RIPPABLE	BLASTING	RIPPABLE	NONE	RIPPABLE	NONE	RIPPABLE	NONE	RIPPABLE	NONE	RIPPABLE	NONE	RIPPABLE
ROCK DITCH	NONE		TRENCHABLE	BLASTING		NONE	TRENCHABLE	NONE	TRENCHABLE	NONE	TRENCHABLE	NONE	TRENCHABLE	NONE	TRENCHABLE
CLEARING	TRENCHABLE					NONE									
BUOYANCY CONTROL	NONE			WEIGHTING			NONE	WEIGHTING	NONE	WEIGHTING	NONE				

INDEX MAP



LEGEND:

PIPELINE ROUTE
CONSTRUCTION STAGING AREA
COMPRESSOR/PUMP STATION
SEISMIC ZONES
SPREAD BOUNDARIES

CLIENT:

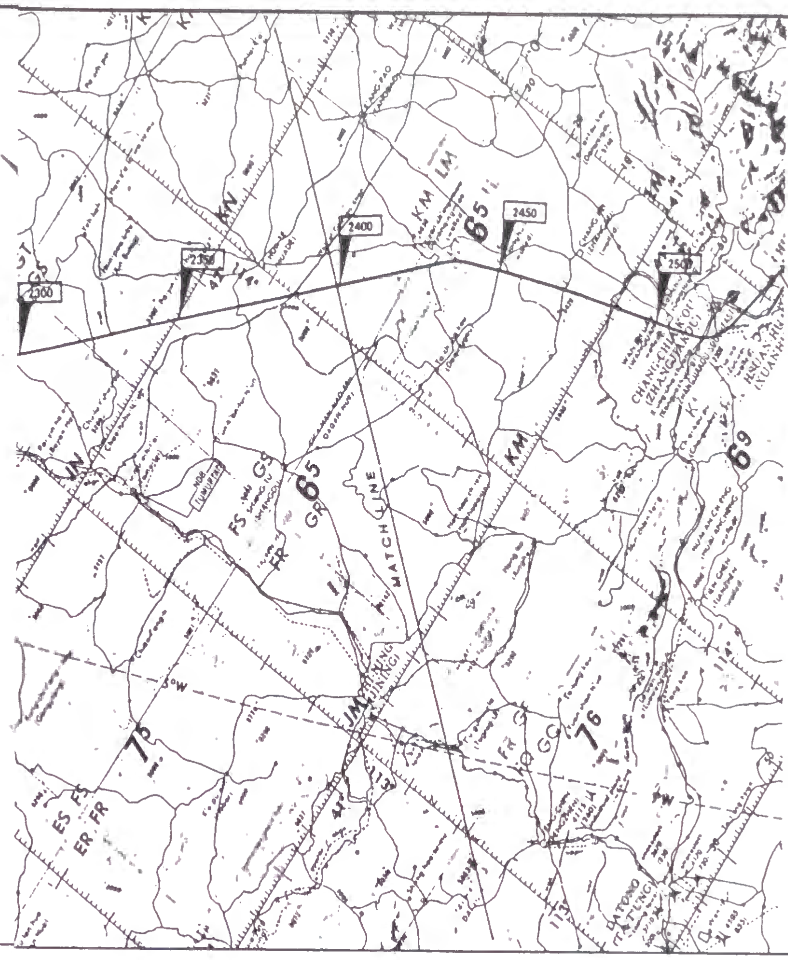
NIPPON STEEL CORPORATION
TOKYO JAPAN

EAST SIBERIA AND FAR EAST PIPELINES

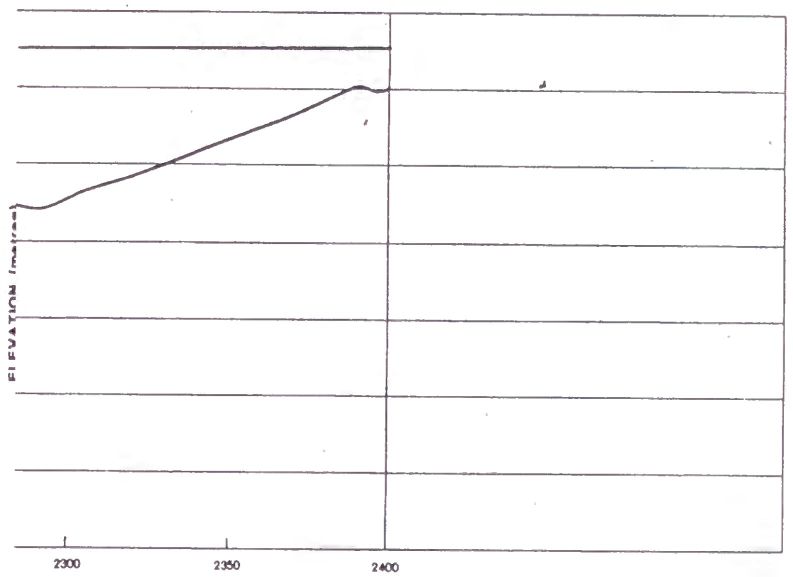
PRE-FEASIBILITY STUDY

DRAFTED:		DATE:	
CHECKED:		JOB NO.:	
SCALE:	AS SHOWN	SHEET NO.:	CG 3 of 5

PLAN



ELEVATION PROFILE



TRAVEL →
SPREAD 14

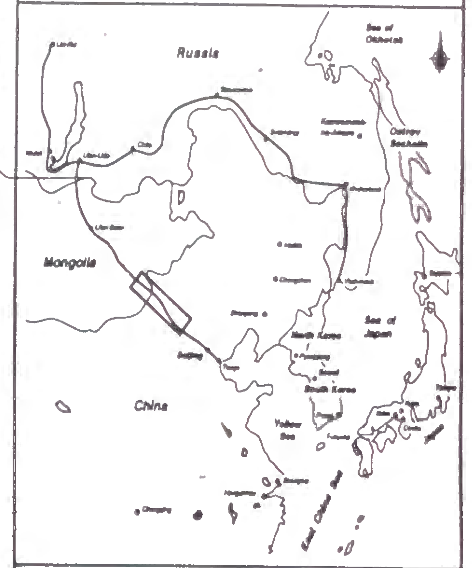
PIPE ℄

KP 2400

TERRAIN AND CONSTRUCTION DATA

VEGETATIO		
SURFICIAL		
PERMAFRO		
ROCK GRA		
ROCK DITC		
CLEARING		
BUOYANCY		

INDEX MAP



LEGEND:

PIPELINE ROUTE

CONSTRUCTION STAGING AREA

COMPRESSOR/PUMP STATION

SEISMIC ZONES

SPREAD BOUNDARIES

CLIENT:

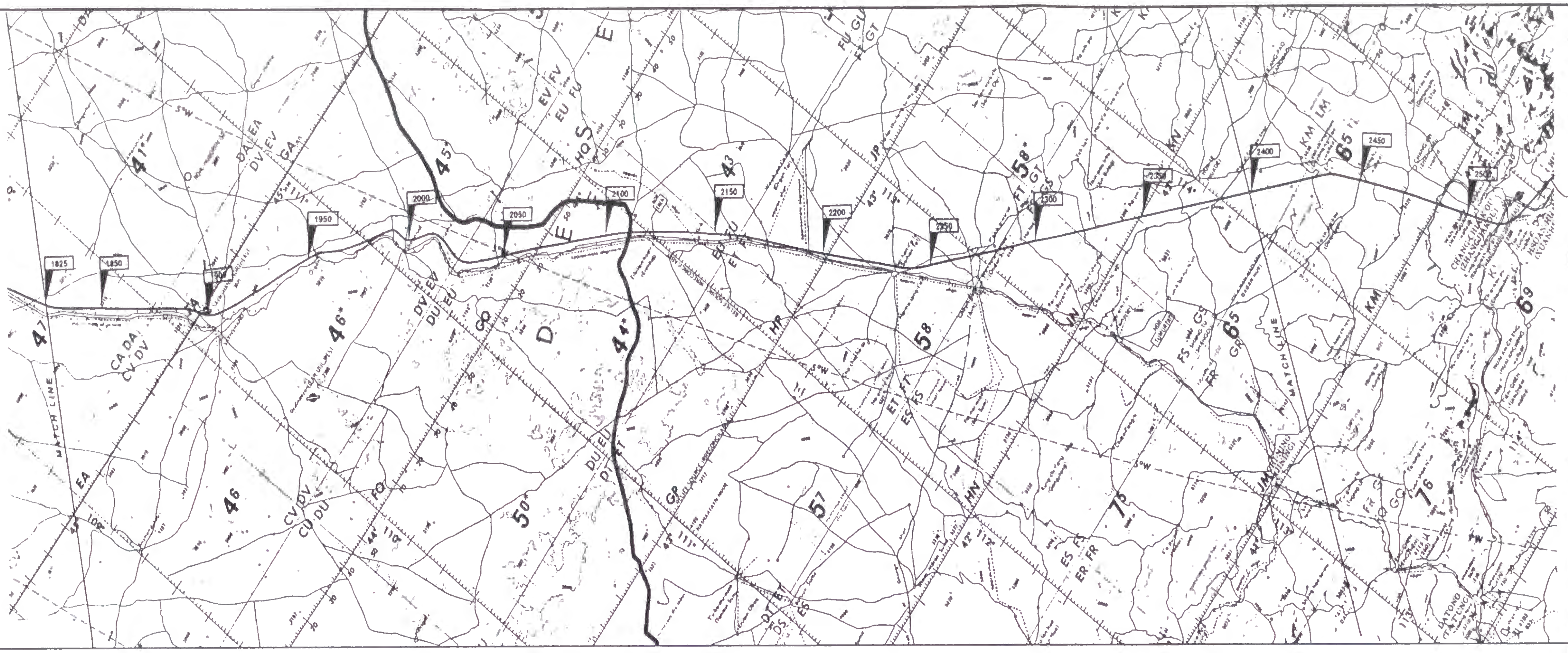
NIPPON STEEL CORPORATION
TOKYO JAPAN

EAST SIBERIA AND FAR EAST PIPELINES

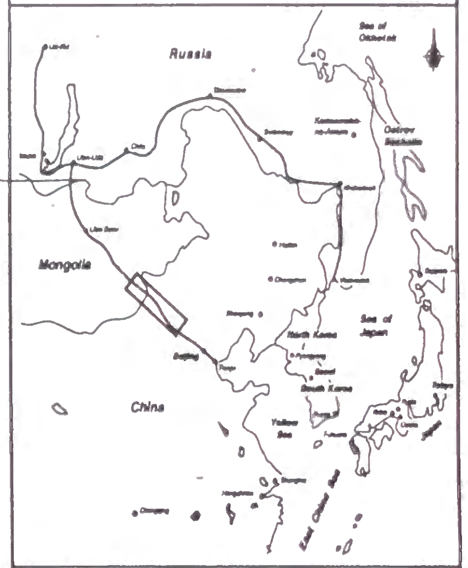
PRE-FEASIBILITY STUDY

DRAFTED:	DATE:
CHECKED:	JOB NO.:
SCALE: AS SHOWN	SHEET NO.: CG 4 of 5

PLAN



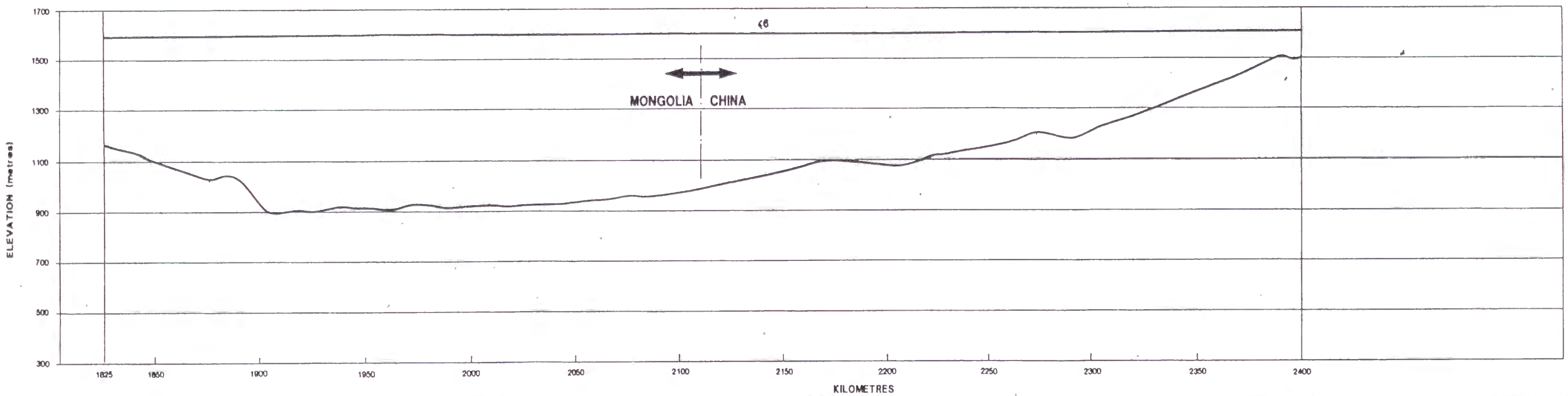
INDEX MAP



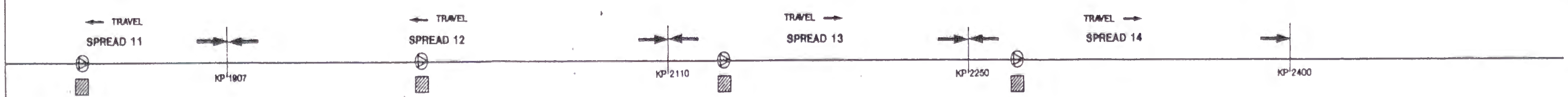
LEGEND:

- PIPELINE ROUTE
- CONSTRUCTION STAGING AREA
- COMPRESSOR/PUMP STATION
- SEISMIC ZONES
- SPREAD BOUNDARIES

ELEVATION PROFILE



PIPE DATA



TERRAIN AND CONSTRUCTION DATA

VEGETATION/LAND USE	FIELD	FIELD	
SURFICIAL GEOLOGY	SOFT ROCK	SHALLOW HARD ROCK	
PERMAFROST/ICE CONT.	NONE		
ROCK GRADE	RIPPABLE	BLASTING	
ROCK DITCH	TRENCHABLE	BLASTING	
CLEARING	NONE	NONE	
BUOYANCY CONTROL	NONE		

CLIENT:

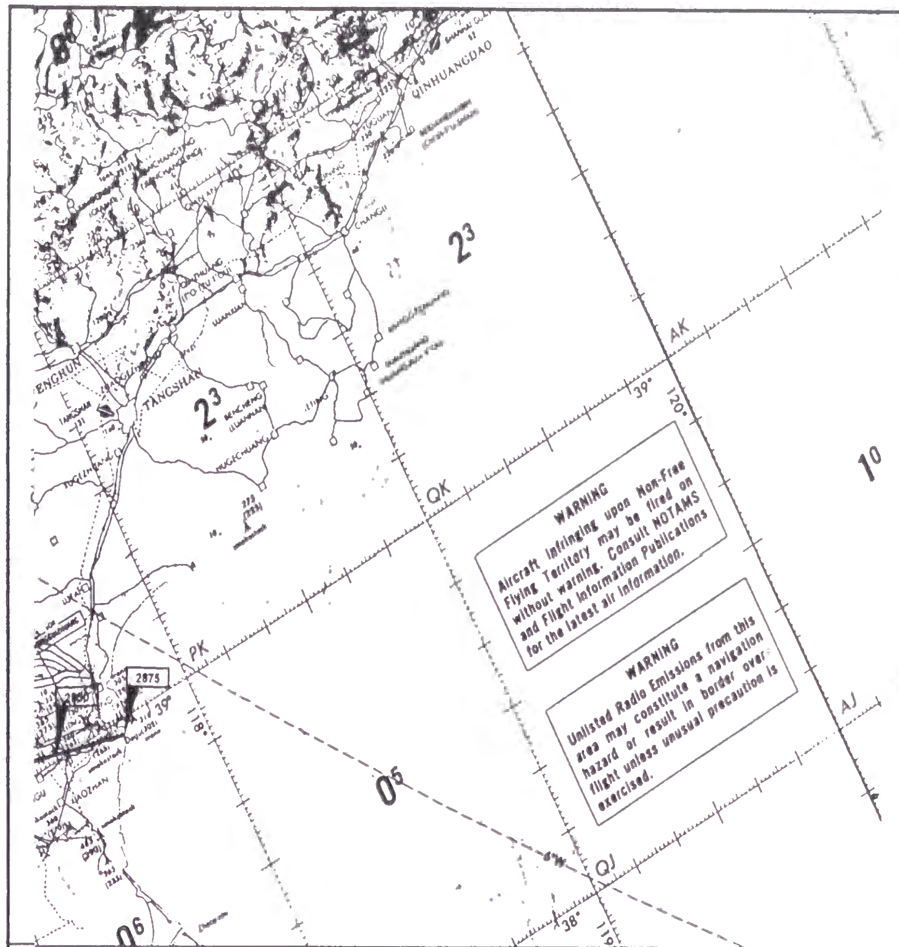
NIPPON STEEL CORPORATION

TOKYO JAPAN

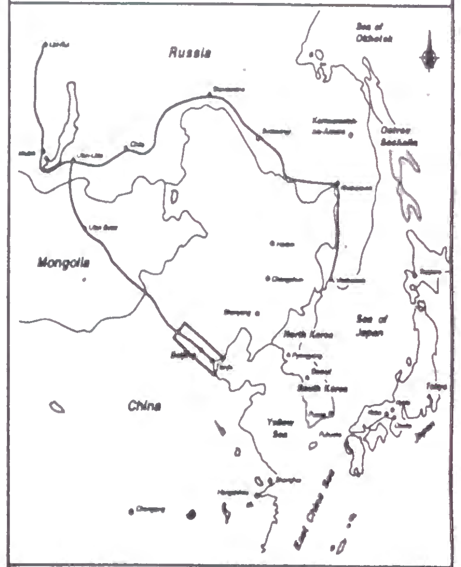
EAST SIBERIA AND FAR EAST PIPELINES

PRE-FEASIBILITY STUDY

DRAFTED:	DATE:
CHECKED:	JOB NO.:
SCALE: AS SHOWN	SHEET NO.: CG 4 of 5

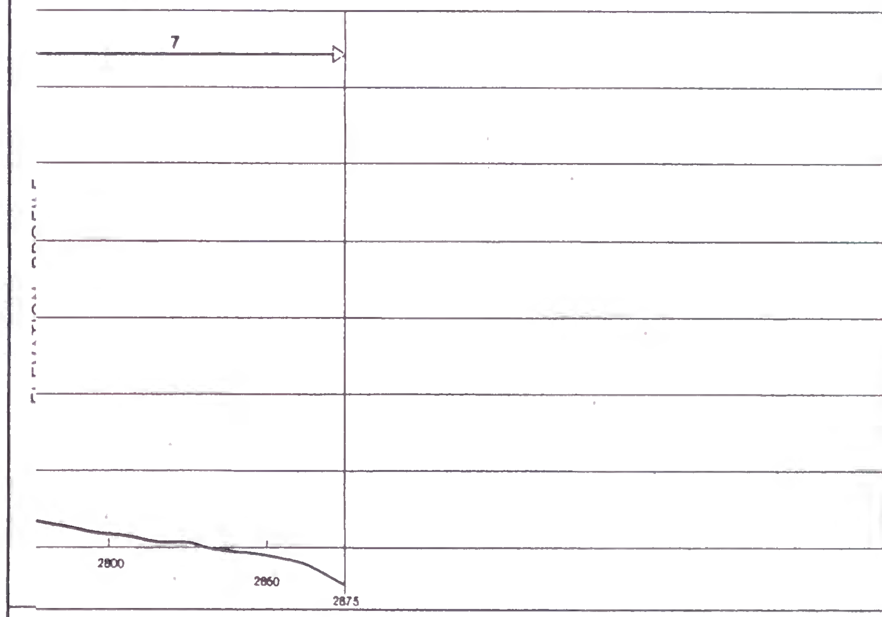


INDEX MAP



LEGEND:

- PIPELINE ROUTE
- CONSTRUCTION STAGING AREA
- COMPRESSOR/PUMP STATION
- SEISMIC ZONES
- SPREAD BOUNDARIES



← TRAVEL

SPREAD 18

2773

KP 2875

TERRAIN AND

	URBAN	VEG PLOT	URBAN
NONE			
NONE			
	NONE	NONE	NONE

CLIENT:

NIPPON STEEL CORPORATION

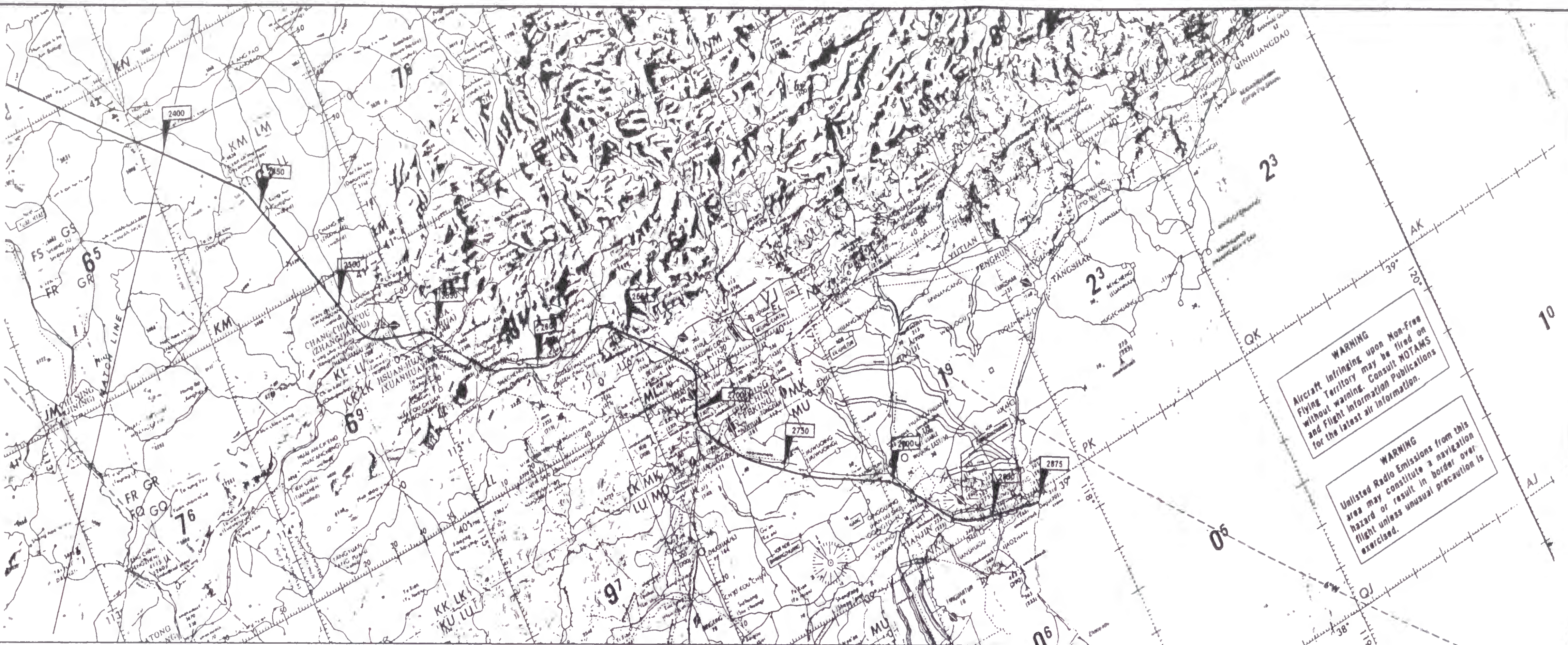
TOKYO JAPAN

EAST SIBERIA AND FAR EAST PIPELINES

PRE-FEASIBILITY STUDY

DRAFTED:	DATE:
CHECKED:	JOB NO.:
SCALE: AS SHOWN	SHEET NO.: CG 5 of 5

PLAN

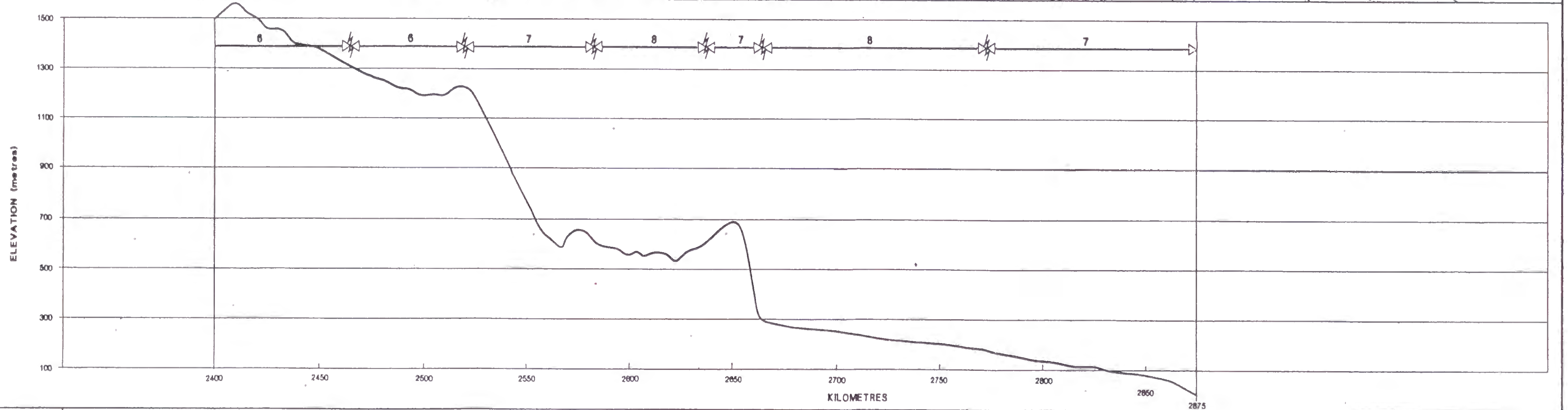


INDEX MAP

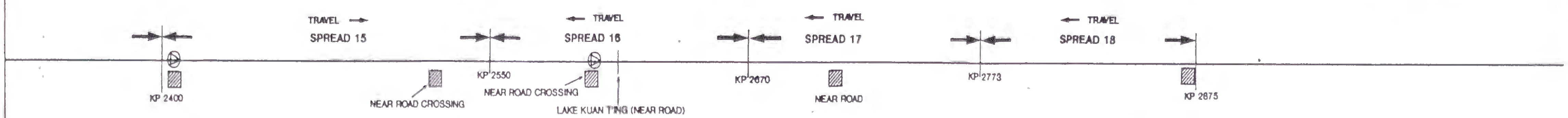
LEGEND:

- PIPELINE ROUTE
- CONSTRUCTION STAGING AREA
- COMPRESSOR/PUMP STATION
- SEISMIC ZONES
- SPREAD BOUNDARIES

ELEVATION PROFILE



PIPE DATA



TERRAIN AND CONSTRUCTION DATA

VEGETATION/LAND USE	FIELD	FOREST	FIELD	SLOPE GRASSLANDS	FIELD	URBAN	VEG PLOT	FIELD	URBAN	VEG PLOT	URBAN
SURFICIAL GEOLOGY	SHALLOW HARD ROCK	SOIL	SOFT AND HARD ROCK	SOIL	HARD ROCK	SOIL	HARD ROCK	SOIL			
PERMAFROST/ICE CONT.	NONE										
ROCK GRADE	BLASTING	NONE	50%BLASTING/50%RIPPABLE	NONE	BLASTING	NONE	BLASTING	NONE			
ROCK DITCH	BLASTING	NONE	50%BLASTING/50%RIPPABLE	NONE	BLASTING	NONE	BLASTING	NONE			
CLEARING	CLEARING	NONE	NONE	NONE	NONE	NONE	NONE	NONE			
BUOYANCY CONTROL	NONE										

CLIENT:

NIPPON STEEL CORPORATION

TOKYO JAPAN

EAST SIBERIA AND FAR EAST PIPELINES

PRE-FEASIBILITY STUDY

DRAFTED:	DATE:
CHECKED:	JOB NO.:
SCALE: AS SHOWN	SHEET NO.: CG 5 of 5

APPENDIX C

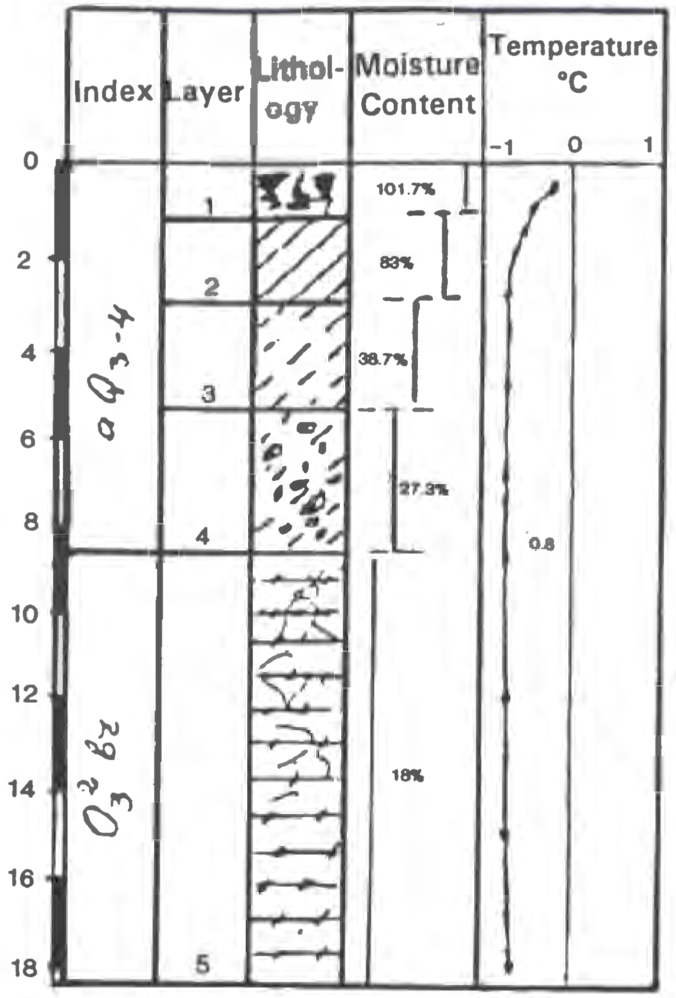
Borehole Logs from Angaro-Lenskiy Region

BH 6007

(Tomgiprotrans, Angara expidition fo Moscow State University)
First terrace of the Tubor River; Right tributary of the Ilim River.

Elevation = 350.0 m

Date of Drilling: 04/1961



Soil/Rock Description

1. Peat.
2. Clayey silt, dark-brown, some peat.
3. Sandy silt.
4. Mixture gravel-cobbles deposits and sandy silt.
5. Argillite with marl seams, weathered.

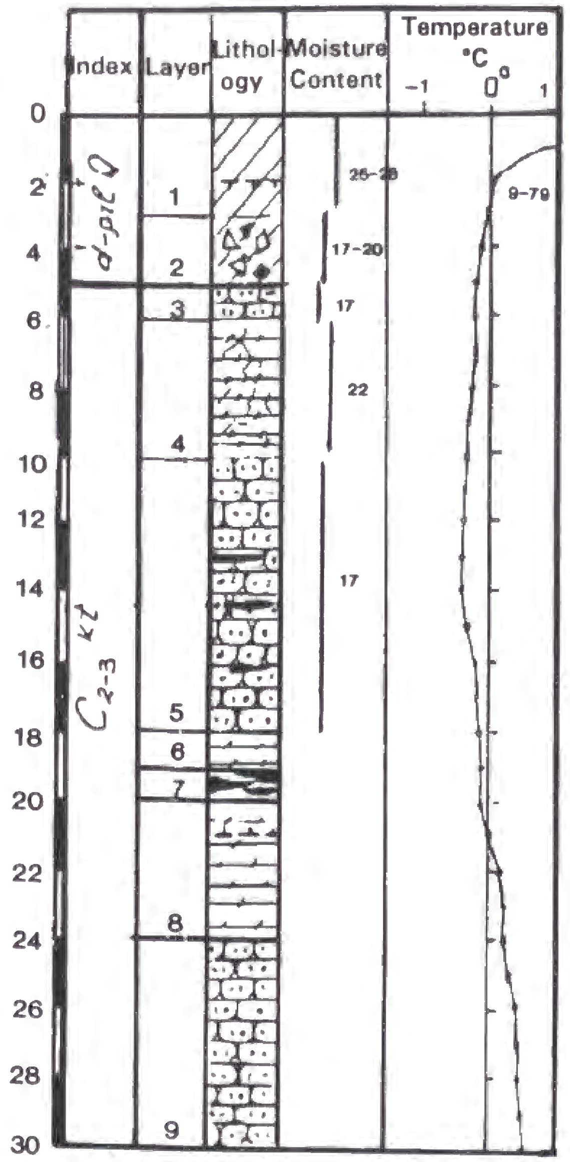
The soils and bedrock are frozen. No description of ice content. According to geophysical data, permafrost thickness is approximately 22 m.

BH 16921

Bottom of the Mostik Creek Valley

Elevation = 271.0 m

Date of Drilling: 06/1977



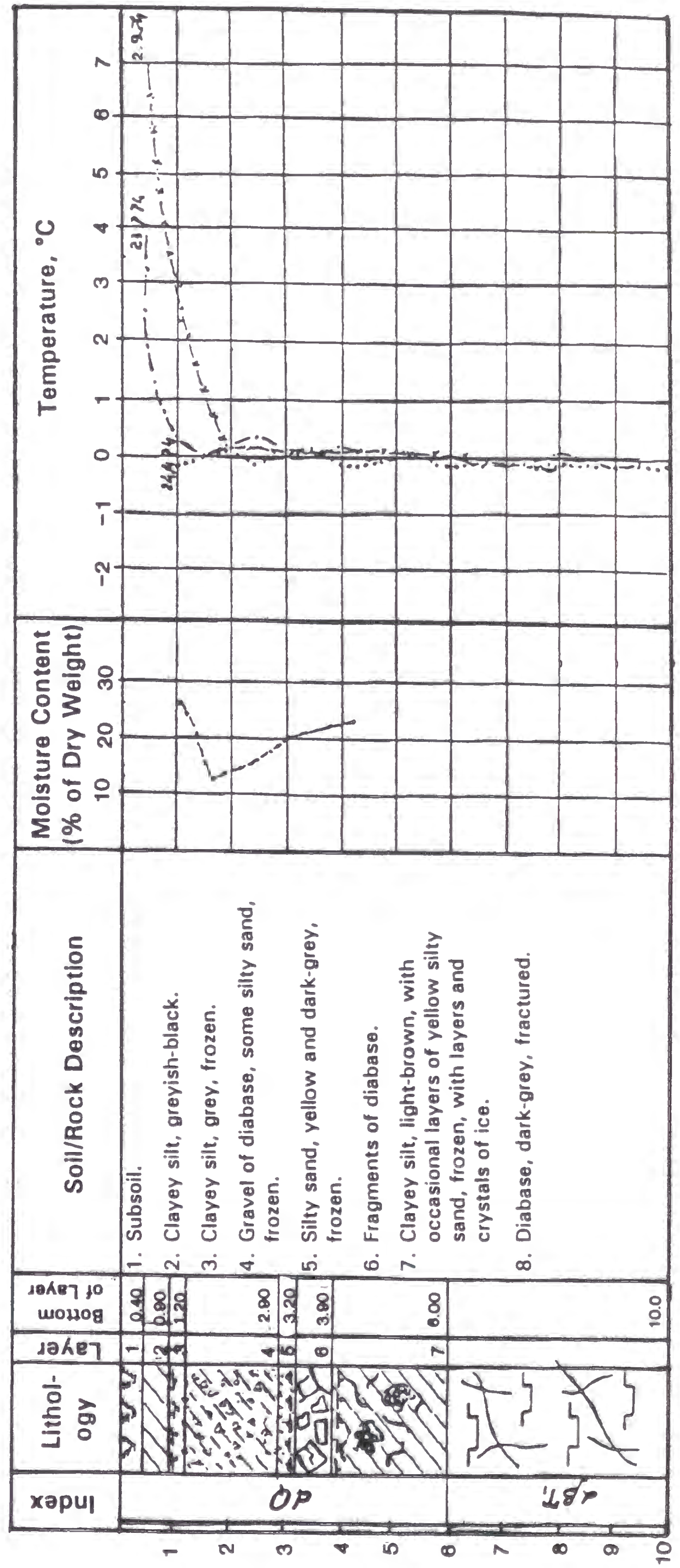
Soil/Rock Description

1. Clayey silt, brown and grey, dense, with iron stains, frozen from 2.0 m. Thin layers of ice.
2. Clayey silt, some gravel of diabase, iron to a considerable extent. Single layers and crystals of ice.
3. Sandstone, fine, greyish-yellow, very weak.
4. Argillite, grey, medium strong, weathered. Single layers of ice, up to 1 mm thick.
5. Sandstone light-grey, fine, weak, with coal seams.
6. Argillite grey, weak.
7. Coal, black, medium strong, weathered.
8. Argillite, grey, medium strong.
9. Sandstone, fine, dark-grey, medium strong.

BH 5750

Hydro Power Station Site

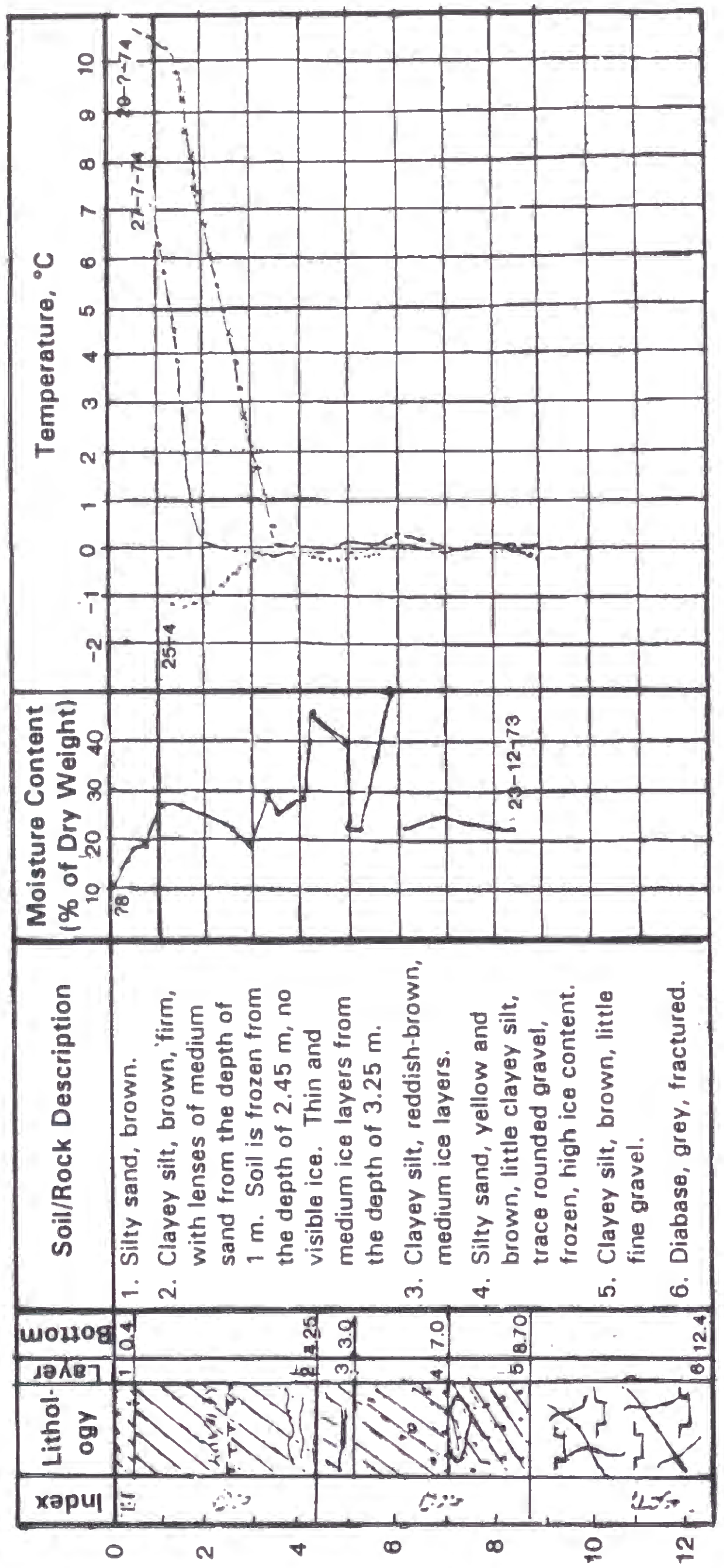
Date of Drilling: 20,21/08/1973



BH NGU 18

Hydro Power Station Site

Date of Drilling: 1973 - 1974

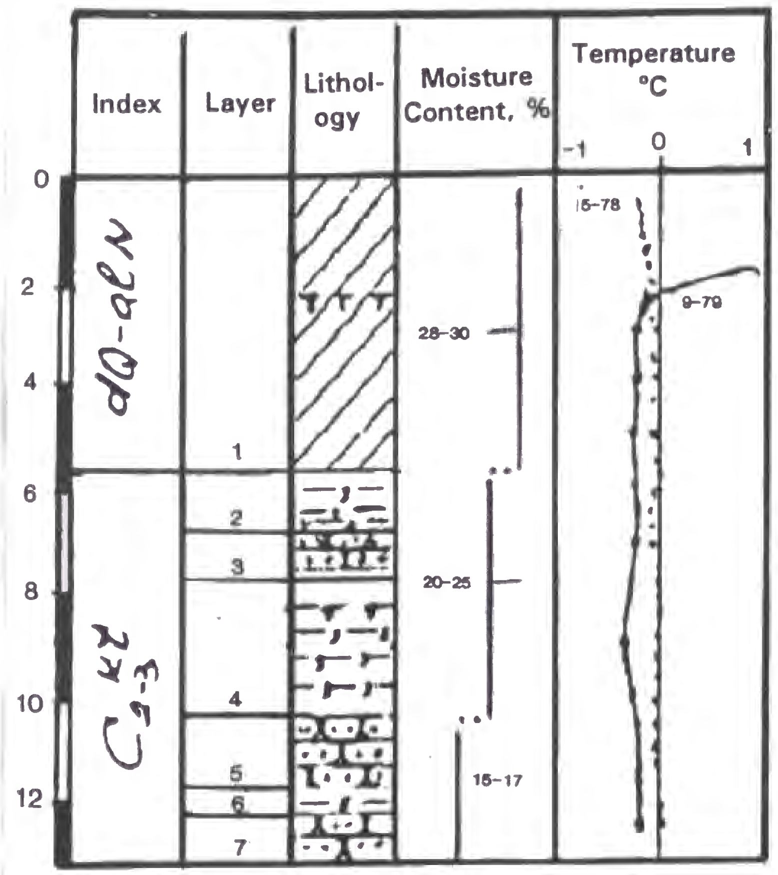


BH 41976

Right bank of the Angara River; Road from Timber Industry Complex to new city.

Elevation = 316.0 m

Date of Drilling: 03/1978



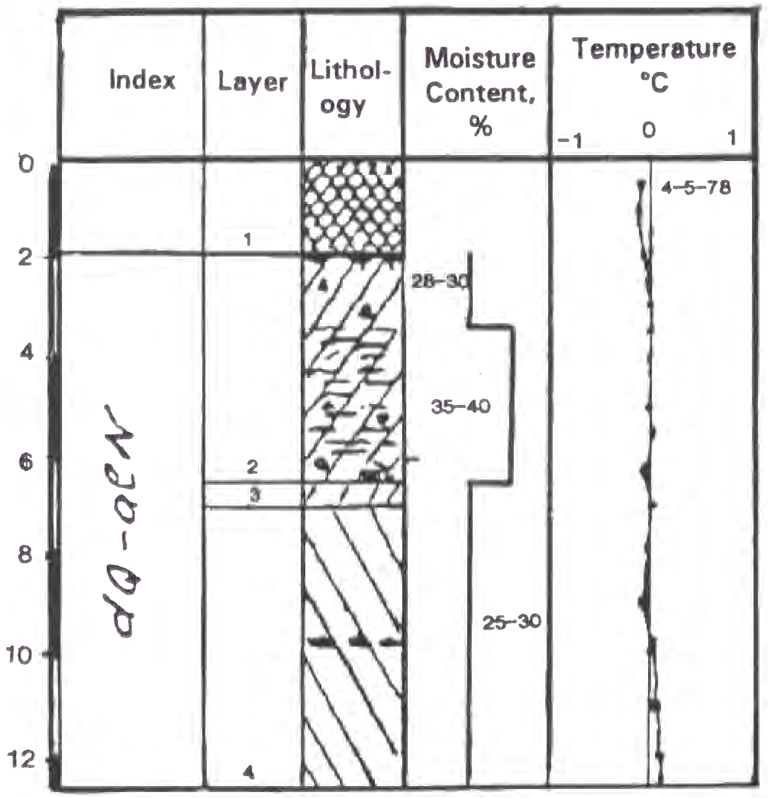
Soil/Rock Description

1. Clayey silt, dark-brown, frozen, no visible ice.
2. Mudstone greenish-grey, with iron stains and layers, frozen, no visible ice, after thawing, weak.
3. Sandstone greenish-grey, medium, frozen.
4. Mudstone greenish-grey, with iron stains, frozen, no visible ice. Mudstone is weak when thawed.
5. Sandstone greenish-grey, medium, frozen, weak after thawing.
6. Argillite, black, frozen, no visible ice.
7. Sandstone, yellowish-grey, medium, strong.

BH 41977

Right bank of the Angara River; Road from Timber Industry Complex to new city.

Elevation = 318.0 m   Date of Drilling: 03/1978



Soil/Rock Description

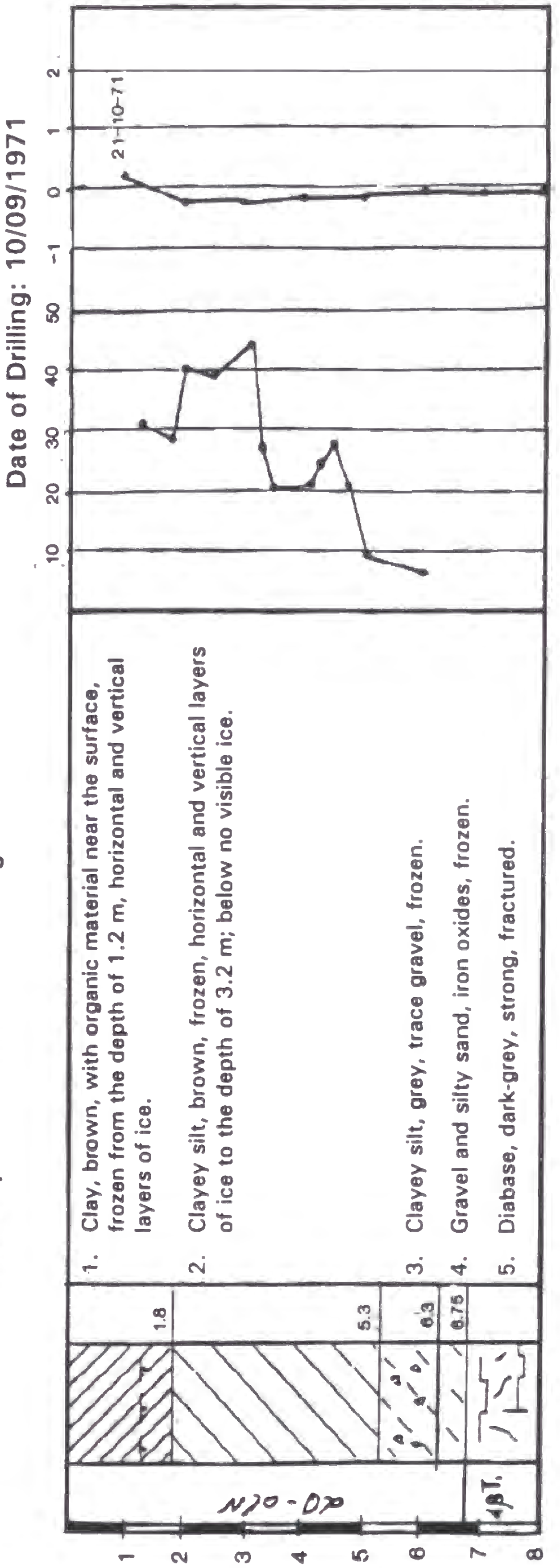
1. Fill, fragments and boulders of diabase, tufa, sandstone, some gravel, frozen.
2. Clayey silt, dark-brown, a little gravel, frozen, no visible ice, with medium ice layers in interval from 3.4 m to 6.5 m.
3. Silty sand, brown with layers of medium sand, frozen, no visible ice.
4. Clay, greenish-grey, frozen to the depth of 9.8 m, no visible ice. After thawing the clay is firm.

Note: Temperature is not steady state. Temperature taken to soon after completion of drilling.

BH 41128

Slope base of the Katykov Valley. Sedge - mossy, birch - spruce forest with dense spruce and fir tree undergrowth.

Date of Drilling: 10/09/1971



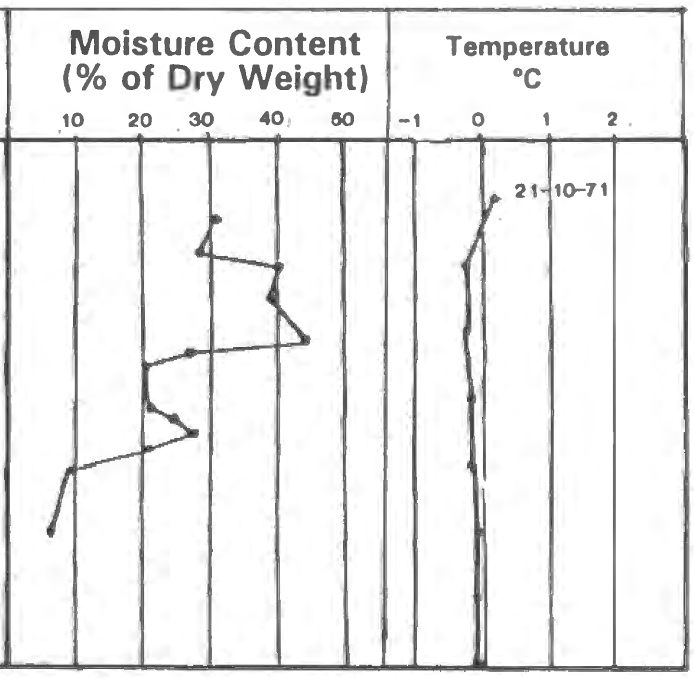
1. Clay, brown, with organic material near the surface, frozen from the depth of 1.2 m, horizontal and vertical layers of ice.
2. Clayey silt, brown, frozen, horizontal and vertical layers of ice to the depth of 3.2 m; below no visible ice.
3. Clayey silt, grey, trace gravel, frozen.
4. Gravel and silty sand, iron oxides, frozen.
5. Diabase, dark-grey, strong, fractured.

BH 41127

Slope base of the Katykov Valley. Sedge - mossy, birch - spruce forest with dense spruce and fir tree undergrowth.

Date of Drilling: 05/09/1971

Index	Bottom of Layer	Soil/Rock Description
dQ - aIN	1.8	1. Clay, brown, with organic material near the surface, frozen from the depth of 1.2 m, horizontal and vertical layers of ice.
	5.3	2. Clayey silt, brown, frozen, horizontal and vertical layers of ice to the depth of 3.2 m; below no visible ice.
	6.3	3. Clayey silt, grey, trace gravel, frozen.
	6.75	4. Gravel and silty sand, iron oxides, frozen.
<βT.		5. Diabase, dark-grey, strong, fractured.

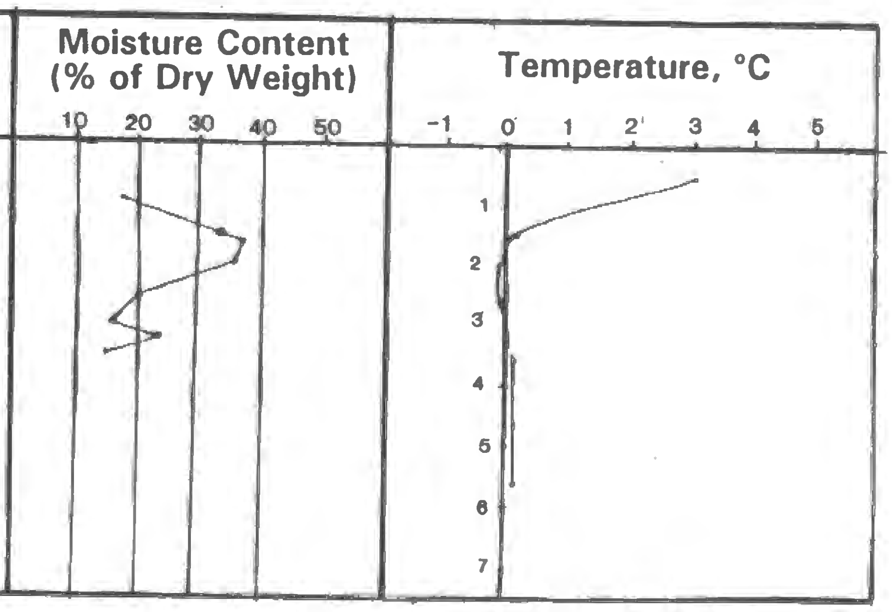


BH 41117

Head of the Katykov Creek. Indications of residual polygon relief. BH is located at relatively high surface. Sedge - mossy, birch - spruce forest with dense spruce undergrowth.

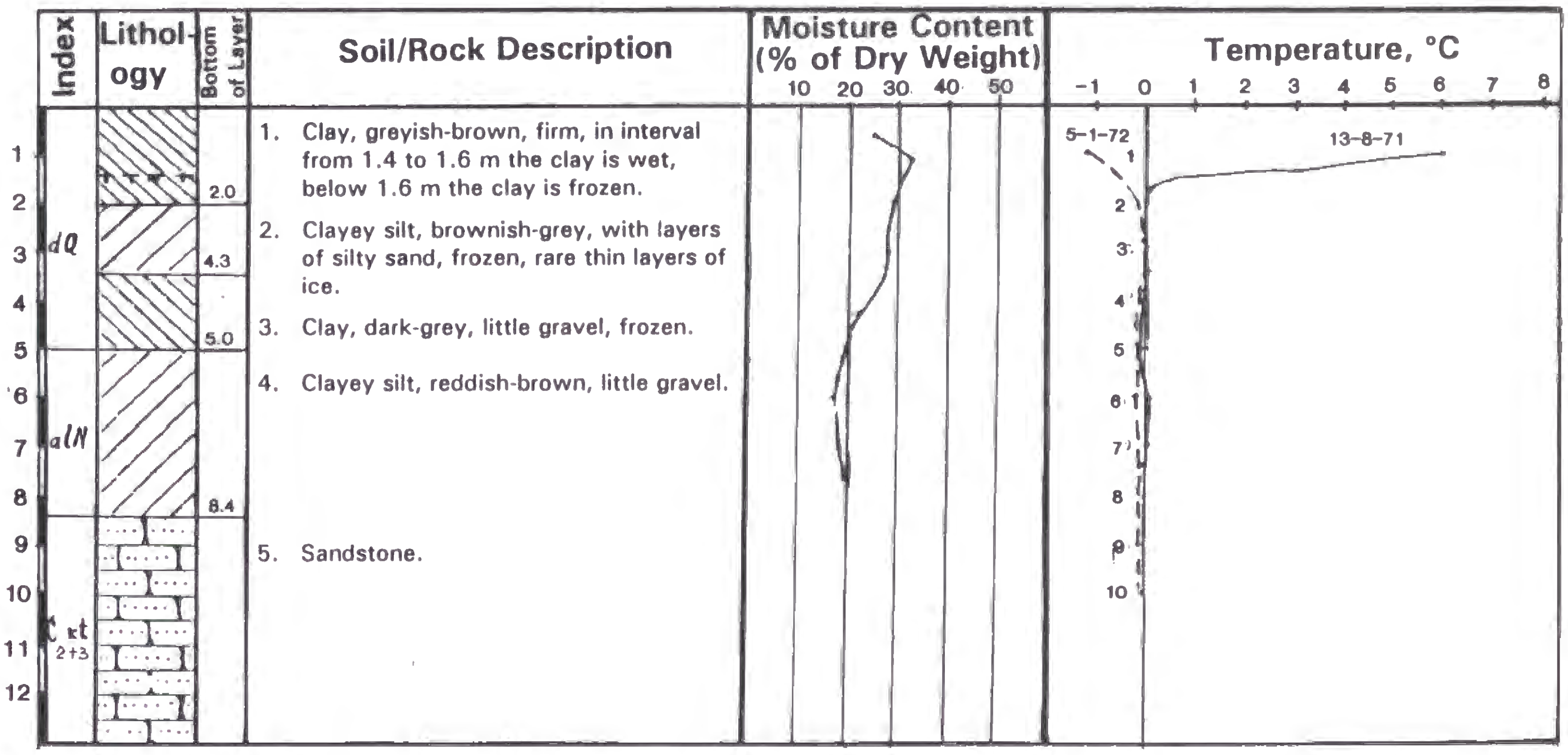
Date of Drilling: 31/08/1971

Index	Bottom of Layer	Soil/Rock Description
dQ	0.8	1. Clayey silt, brown, crumpled.
aIN	2.3	2. Clay, greyish-yellow, frozen from the depth of 1.4 m. Ice crystals to the depth of 1.7 m; below ice layers, thin and medium.
	3.2	3. Clayey silt, brown, with thin layers of silty sand, little gravel, occasional thin layers of ice.
	3.6	4. Sand, yellowish-grey, medium, frozen, no visible ice.
<β1		5. Diabase.



BH 40105

Gentle north-west facing slope. Sedge - mossy, two level forest. (first level - mixed forest; second level - spruce and fir - tree forest).

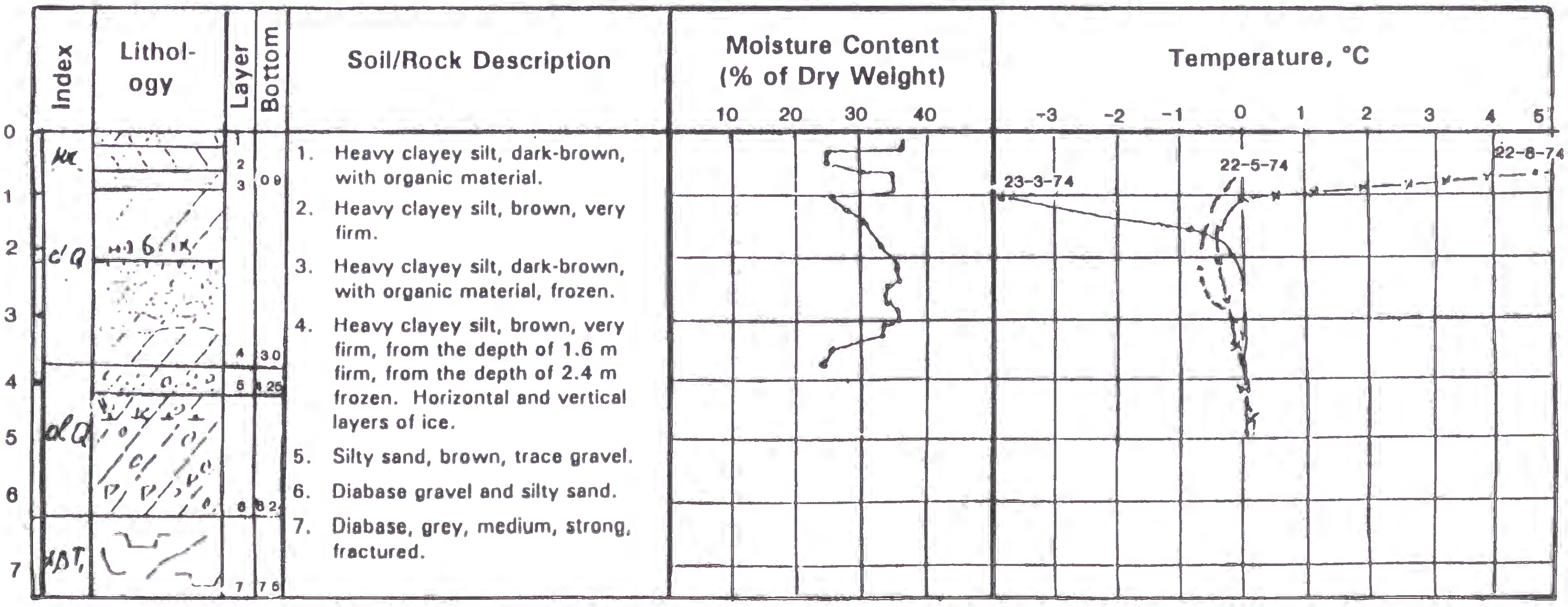


C10

BH of Moscow State University No. 29

The Site of the Timber Industry Complex, Main Building

Date of Drilling: 1974

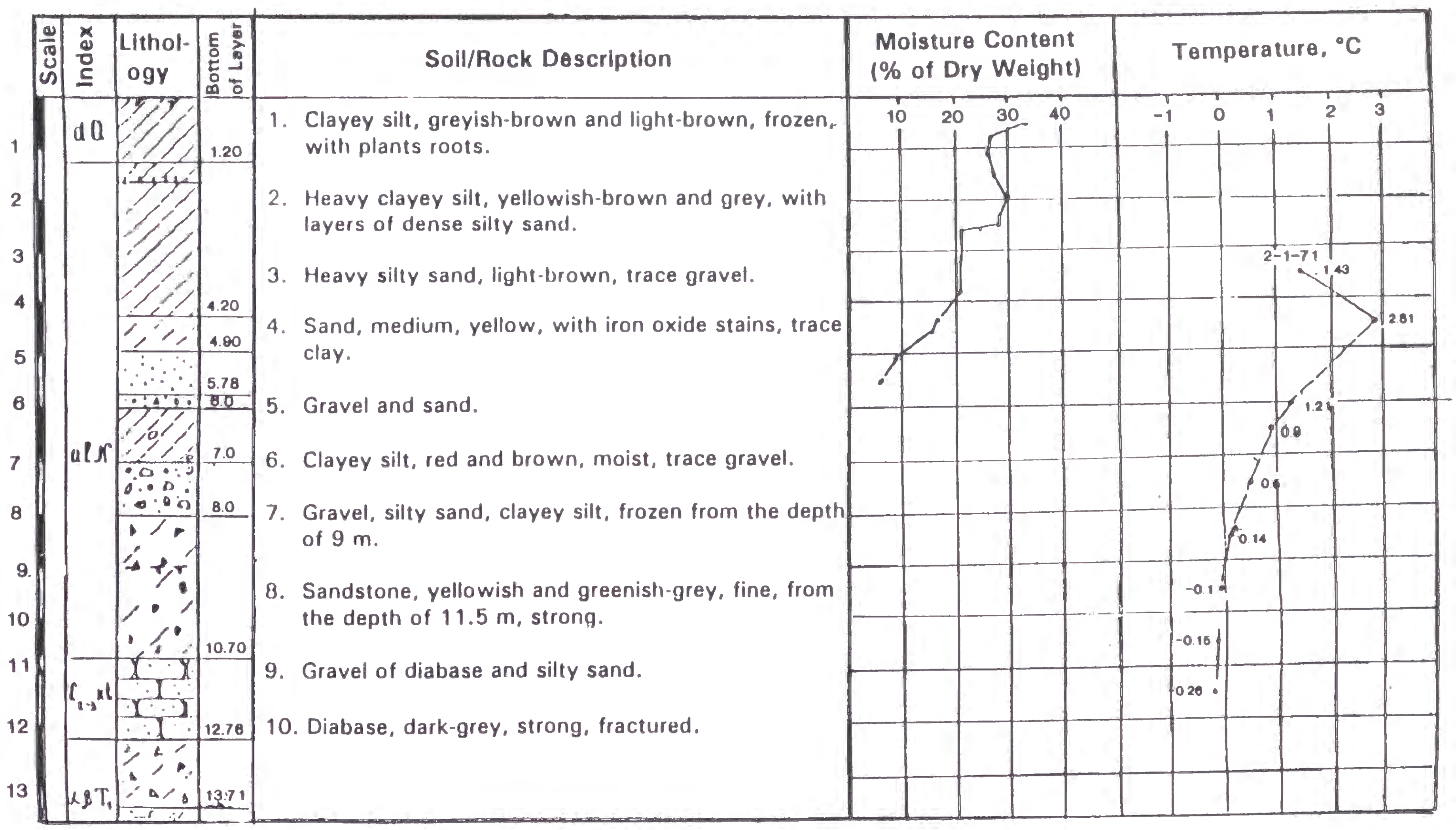


C11

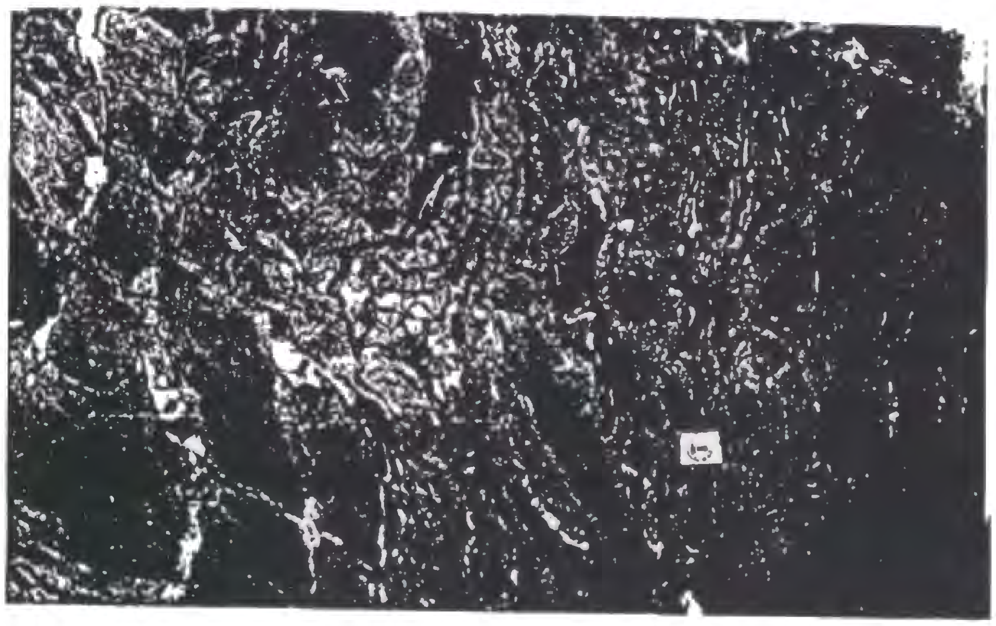
BH 41204

Elevation = 344.0m

Date of Drilling: 05/04/1972



C12



C13: Ice lens structure in clayey soil